ESTIMATION OF FIRE DAMAGE OF REINFORCED CONCRETE BEAMS

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ABSTRACT

A mathematical model for estimating structural performance of reinforced concrete beams exposed to fire has been set up by combining two mathematical submodels for calculation of temperature and structural performance. And the validity of the model has been verified by experiments on two dimensions of full scale beams. For listing the fire damage of 720 types of beams with different specifications such as dimensions, total amount and compressive/tensile cross-sectional area ratio of reinforcing steel bars to be included in the beams, the mathematical model has been applied. Some of the results are presented as examples in this paper.

INTRODUCTION

To cope with the rehabilitation of a reinforced concrete structure which has been exposed to fire, it has been often needed to estimate the fire damage or residual strength of the structural members. As methods for the estimation, the following field tests on the fire exposed reinforced concrete members at the fired building have been usually applied so far in Japan :

- (1) measurement of depth of chemical neutralization of the concrete by use of phenolphthalein (estimation of the heat influenced depth from fire exposed surface of the concrete) ;
- (2) measurement of decrease of elasticity of the concrete by means of the Schmidt Rebound Hammer ;
- (3) observation of color change of the concrete with eyes ;
- (4) measurement of flexural rigidity of the structure composed of beams and floor slabs by loading test.

The test methods (1)~(3) are available only for rough estimation, and (4) is not easy in loading on the structure and measuring the deflection of it in practice. Such experimental estimation methods in the field as mentioned above have inadequacy in terms both of reliability and workability. Therefore, a new more reliable and workable method alternative to these traditional methods has been desired to be developed.

As we have already basic theories available for the development of a new mathematical model for estimating structural damage by fire, necessary approach for us to attempt is to systematize those theories, to verify the validity of the model through experiments. Then it will be very convenient in actual practice to provide a list for knowing easily the degree of the structural damage of the fire exposed beams, which can be made by applying the mathematical model. A beam is generally apt to be damaged in fire more severely than any other members, ac-

cordingly the beam is only taken up in this paper as a member objective for derivation and discussion of the mathematical model, verification of the model by the experiments and preparation of the list of the structural performance of fire exposed beams with various specifications.

FRAMEWORK OF MATHEMATICAL MODEL FOR FIRE DAMAGE ESTIMATION

The mathematical model for estimating the damage of reinforced concrete beams exposed to fire is to be fundamentally composed by the following three calculation submodels :

- (1) on fire severity (fire temperature-time curve) in the fire compartment ;
- (2) on temperature history in the beam heated by the fire whose severity can be given by the model (1) ;
- (3) on residual structural performance of the beam suffered through the elevated temperature history obtained by the model (2).

The outline of each submodel is briefly described in the followings.

Estimation of Fire Severity

At the first step of the calculation procedure, the fire severity should be given as the thermal load to act on the objective beam during fire. The thermal load can be expressed by a temperature-time curve for the fire compartment where the surfaces of the beam are heated. And this is to be used as a premise condition in the next step on the calculation of the temperature history. As known in general, the model available for this estimation has been already developed by K.Kawagoe¹⁾ and others. The following input data are necessary for this process : thermal properties of materials of surrounding structural members, dimensions of the fire compartment and its openings, amount of the combustible in the fire compartment and so on.

Estimation of Temperature History

As the second step of the procedure, the temperature history should be calculated to

know the maximum temperature elevated at each part in the cross-section of the beam. The thermal load acting to the fire-exposed surfaces of the beam is to be given as a function of the fire temperature which is obtained through the first step of the procedure mentioned at the last. The model of the temperature history of each part inside the beam is to be calculated by a finite difference method taking into account the effect of absorption of latent heat by moisture vaporization. The following input data are necessary for this process : dimensions of the cross-section of the beam, moisture content, thermal properties of the materials composing the beam.

Estimation of Residual Performance of Fire Exposed Beam

As the third or last step of the procedure, the strength and rigidity of the beam whose temperature history has been given at the second step should be calculated to estimate the fire damage or residual structural performance of the beam exposed to fire. When strength and elastic modulus of the materials composing the beam are given as functions of the maximum temperature elevated by fire exposure, the structural performance such as the strength and rigidity can be calculated applying the relevant calculation method which is generally used for room temperature and derived by making the stress balance for the cross-section. The following data are necessary for this process : dimensions of the cross-section, mechanical properties of materials composing the beam as functions of the maximum temperature elevated by fire exposure.

As being already generalized and able to be replaced by the standard fire temperature curve such as described in JIS (Japanese Industrial Standards), discussions for the fire severity model is to be omitted in the later section.

CALCULATION OF TEMPERATURE OF BEAM WITH RECTANGULAR SECTION

Fig.1 Rectangular network of elements in a half section of beam

As it will be assumed that the heat flow along the longitudinal axis of the beam can be neglected in such problems, the calculation model may be made for two-dimensional heat flow for a cross-section at right angles to the axis as shown in Fig.1, using a numerical method or finite difference calculation method.

The temperature increment $\Delta\theta$ per a time step Δt for an arbitrary element (i, j) in the rectangular network model where the cross-section is subdivided into a number of elements as shown in Fig.1, is obtained from Eq.(1) derived by making heat balance for the elements, where $H_1 \sim H_4$ are given in table 1.

With the aid of Eqs.(1) to (6) and the other data such as the relevant material properties described above, the temperature history for each element can be calculated. A temperature for an element can be obtained for any time $[t=(n+1)\Delta t]$ as the sum of the temperature at the time $[t=n\Delta t]$ and $\Delta\theta$.

$$\Delta\theta(i, j) = \frac{\Delta t}{C_p} \left\{ H_1[\theta(i-1, j) - \theta(i, j)] + H_2[\theta(i+1, j) - \theta(i, j)] + H_3[\theta(i, j-1) - \theta(i, j)] + H_4[\theta(i, j+1) - \theta(i, j)] \right\} \dots (1)$$

Table 1 Coefficient in Equation (1)

range of i	range of j	H ₁	H ₂	H ₃	H ₄	remarks
i = 1	j = 1	2H _x ^h	2H _x ^c	2H _y ^h	2H _y ^c	θ(i-1, j) = θ _F θ(i, j-1) = θ _F
	2 ≤ j ≤ n-1	2H _x ^h	2H _x ^c	H _y ^c	H _y ^c	θ(i-1, j) = θ _F
	j = n	H _{1n} ¹	H _{2n} ¹	H _{3n} ¹	H _{4n} ²	θ(i-1, j) = θ _F
	n+1 ≤ j ≤ s-1	0	H _x ^c	H _y ^c	H _y ^c	
	j = s	0	H _x ^c	2H _y ^c	2H _y ^o	θ(i, j) = θ _F
2 ≤ i ≤ m	j = 1	H _x ^c	H _x ^c	2H _y ^h	2H _y ^c	θ(i, j-1) = θ _F
	2 ≤ j ≤ n-1	H _x ^c	H _x ^c	H _y ^c	H _y ^c	
	j = n	H _{1n} ²	H _{2n} ²	H _{3n} ²	H _{4n} ²	
	n+1 ≤ j ≤ s-1	H _x ^c	H _x ^c	H _y ^c	H _y ^c	
	j = s	H _x ^c	H _x ^c	2H _y ^c	2H _y ^o	θ(i, j+1) = θ _F

where

$$H_x^h = \frac{h_F}{\Delta x}, \quad H_x^c = \frac{\lambda}{\Delta x^2}$$

$$H_y^h = \frac{h_F}{\Delta y}, \quad H_y^c = \frac{\lambda}{\Delta y^2}$$

$$\bar{H}_x^c = \frac{\bar{\lambda}}{\Delta x^2}, \quad \bar{H}_y^c = \frac{\bar{\lambda}}{\Delta y^2}$$

$$\bar{H}_y^o = \frac{h_o}{\Delta y}$$

$$H_{1n}^1 = [h(\Delta x + \Delta y) + \frac{\lambda \Delta y}{\Delta x}] / (\Delta x \cdot \Delta y)$$

$$H_{2n}^1 = (\bar{\lambda} + \lambda)(\Delta y + \Delta \bar{y}) / (2\Delta x^2 \Delta Y)$$

$$H_{3n}^1 = \lambda / (\Delta y \cdot \Delta Y), \quad H_{4n}^1 = 2\bar{\lambda} / (\Delta \bar{y} \cdot \Delta Y)$$

$$H_n^2 = (\bar{\lambda} + \lambda)(\Delta y + \Delta \bar{y}) / (2\Delta x^2 \cdot \Delta Y')$$

$$H_{3n}^2 = 2\lambda(\Delta y \cdot \Delta Y'),$$

$$H_{4n}^2 = 2\bar{\lambda}(\Delta y \cdot \Delta Y')$$

where

$$\Delta Y = \frac{\Delta y}{2} + \frac{\bar{C}_p}{C_p} \Delta \bar{y}, \quad \Delta Y' = \Delta y + \frac{\bar{C}_p}{C_p} \Delta \bar{y}$$

θ_F: fire temperature given as a function of time "t"

h_F: heat transfer coefficient at fire exposed sur-

face of the beam

h_o: heat transfer coefficient at surface of floor slab on the beam

(the floor slab faces to colder space upper than the fire compartment)

λ, λ̄: heat conductivity of the materials composing the beam, the floor slab respectively,

C_p, C̄_p: specific heat per unit volume of the materials composing the beam, the floor slab respectively,

Adiabatical condition or θ(i+1, j) = θ(i-1, j) is assumed for the plane i=1 in the range of n ≤ j ≤ s and the plane of symmetry i=m.

Specification for the Calculation of Temperature

1) Fire Temperature

As the fire temperature course to be used for the calculation, the standard temperature-time curve prescribed in JIS-A-1304 is applied, which corresponds almost with the ISO standard curve for fire resistance test of structural members and can be approximately described by the following expression Eq.(2):

$$\theta_F(t) = 1080 - 340e^{-0.8t} - 130e^{-5t} - 610e^{-19t} \quad \dots\dots\dots(2)$$

where t : the time in hours

2) Effect of Moisture to Heat Transmission

The effect of moisture is taken into account in the calculation by assuming that the moisture starts to evaporate when the temperature of the relevant element reaches 100°C and continues to absorb the latent heat for evaporation until all the moisture in the element is exhausted. It is also assumed that the moisture does not produce any other effects on heat transmission. Therefore, the moisture effect can be treated as the apparent heat or temperature alternative to the latent heat in the calculation process. The total temperature decrease by the latent heat or moisture evaporation an unit volume $\Delta\theta_m$ is given by :

$$\Delta\theta_m = 539 w/c \quad \dots\dots\dots(3)$$

where w : moisture content per unit volume of the concrete,
 c : specific heat of the concrete.

3) Heat Transfer Coefficient from Fire to the Beam Surface

The heat transfer from fire to the exposed surface of the beam can be divided into the following two factors ; one is the heat radiation from flame or combustion products, surfaces of burning substances and surroundings of the fire compartment etc., and the other is the heat convection by the flame or gas flow.

The radiative heat transfer coefficient h_R can be written as ;

$$h_R = 4.88 \times 10^{-8} \epsilon_w (T_F + T_W) (T_F^2 + T_W^2) \quad \dots\dots\dots(4)$$

where emissivity of fire ϵ_F is assumed to be an absolute black body or $\epsilon_F = 1$

ϵ_w : emissivity of the exposed surface of the

beam

T_F, T_W : absolute temperature of the fire, the exposed surface respectively

The convective heat transfer from fire gas to the exposed surface depends on the flow condition and physical properties of the gas. The convective heat transfer coefficient h_c for turbulent flow parallel to a flat plate is adopted here. For the turbulent flow, when the "j" factor by Colburn is defined by the following equation ;

$$j = \frac{h_c}{\rho U_f C_p} \left(\frac{\mu C_p}{\kappa} \right)^{3/2} = \frac{0.037}{(\ell U_f \rho / \mu)^{1/5}}$$

there is good agreement between the j factor and a friction coefficient and then the convective heat transfer coefficient h_c is given by :

$$h_c = \frac{1300 C_p (\rho U_f)^{4/5}}{Pr^{2/3}} \left(\frac{\mu}{x} \right)^{1/5} \quad \left[\frac{\text{Kcal}}{\text{m}^2 \text{hr } ^\circ\text{C}} \right]$$

where C_p , ρ , μ , κ , Pr and U_f denote specific heat at constant pressure [Kcal/Kg $^\circ\text{C}$], density [Kg $\cdot\text{s}^2\cdot\text{m}^4$], viscosity coefficient [Kg $\cdot\text{s}/\text{m}^2$], thermal diffusivity [m $^2/\text{hr}$], Prandtl Number and flow velocity [m/s] respectively, and x is the distance from an edge whose co-ordinates in the cross-section is shown as (i, j)=(1,1). When the representative length " ℓ " for the relevant system of flow is given, then the average value of the convective heat transfer coefficient $\bar{h}_c$ in the range of $0 \leq x \leq \ell$ can be approximately given as :

$$\bar{h}_c = h_c \times \frac{5}{4} \left(\frac{x}{\ell} \right)^{1/5}$$

The total heat transfer coefficient for the fire exposed surface h_F can be given as the sum of h_R and h_c , that is ;

$$h_F = 4.88 \times 10^{-8} \epsilon_w (T_F + T_W) (T_F^2 + T_W^2) + \frac{1625 C_p (\rho U_f)^{4/5}}{Pr^{2/3}} \left(\frac{\mu}{\ell} \right)^{1/5} \quad \dots\dots\dots(5)$$

4) Heat Transfer Coefficient on the Back Surface

The heat transfer on the back surface of the floor slab facing to the adjacent colder space at room temperature is also divided into the radiative and free convective heat transfer. For the radiative heat transfer coefficient h_{ro} , Eq.(4) can be used by substituting the emissivity, the temperature of the back surface of the member and the temperature of the adjacent space into ϵ_w , T_w , and T_F respectively.

For the free convection heat transfer coefficient h_{co} on a horizontal surface, the following equations can be given :

$$\text{for } 10^5 < Gr \cdot Pr \leq 2 \times 10^7 : h_{co} = 1.1(\Delta T / \ell)^{1/4}$$

$$Gr \cdot Pr > 2 \times 10^7 : h_{co} = 1.3\Delta T^{1/3}$$

where

Gr : Grashof Number for heat transfer,

Pr : Prandtl Number,

ΔT : temperature difference between the back surface of the floor slab and the colder space where the surface is facing.

When we give $\ell = 3\text{m}$ for the value of the representative length, then :

$$Gr \doteq 1.7 \times 10^{11}$$

$$\therefore Gr \cdot Pr \doteq 1.2 \times 10^{11} > 2 \times 10^7$$

$$\therefore h_{co} = 1.3\Delta T^{1/3}$$

In consequence, the total heat transfer coefficient on the back surface h_o is obtained by ;

$$h_o = 4.88 \times 10^{-8} \epsilon_b (T_b + T_o) (T_b^2 + T_o^2) \\ + 1.31 |T_b - T_o|^{1/3} \dots\dots\dots (6)$$

Premise Condition required for Time Step in the Calculation

In order to ensure stable solutions in the calculations, the maximum of the time step Δt is limited to satisfy the criterion shown by the following inequality on the basis of the heat balance for each element meshed in the cross-section:

$$\frac{\Delta t}{Cp} \sum_{i=1}^4 H_i \leq 1$$

where H_i is, as shown in Table 1, different in each element (i, j). As for the model shown in Fig.1. for example, the value of $\sum H$ is maximum at the element (i, j)=(1, 1). Therefore we should decide the value of Δt so that the inequality can be satisfied at the element. As the result, we can get a common time step Δt for this calculation model from the following inequality:

$$\Delta t \leq 0.5 C_p \{(\Delta x^{-1} + \Delta y^{-1})h_F + (\Delta x^{-2} + \Delta y^{-2})\lambda\}^{-1}$$

As the the value required for Δt is different in each element as mentioned above, we can reduce calculation time by adopting the maximum value of Δt for each element. At least, it is not necessary to adopt only the most restrictive value of Δt as a common value described above through an entire calculation.

CALCULATION OF STRUCTURAL PERFORMANCE OF REINFORCED CONCRETE BEAM EXPOSED TO FIRE

When a reinforced concrete beam is heated by fire, its internal temperature history is decisively influenced by the depth from the fire exposed surfaces of the beam. The structural performance such as strength and rigidity of the beam may be decreased according to the temperature history or maximum temperature distribution of the beam. The mathematical model for calculating the structural performance, that is, bending strength and initial rigidity, can be derived from the stress balance equation for the subdivided rectangular network of the cross-section shown in Fig.1. The calculation through this model is to be done by inputting the mechanical properties of the materials which are given as functions of the temperature history. The stress intensities of the respective parts of the concrete in compressive side, the reinforcing bars in compressive side and those in tensile side denoted as $\sigma_{cc}(i, j)$, $\sigma_{sc}(i, j)$, $\sigma_{st}(i, j)$ are given by:

$$\left. \begin{aligned} \sigma_{cc}(i, j) &= \frac{y}{n} \frac{E_{cc}(i, j)}{E_n} \sigma_n \\ \sigma_{sc}(i, j) &= \frac{n - d_c}{n} \frac{E_{sc}(i, j)}{E_n} \sigma_n \\ \sigma_{st}(i, j) &= \frac{d - n}{n} \frac{E_{st}(i, j)}{E_n} \sigma_n \end{aligned} \right\} \dots \dots \dots (7)$$

[NOMENCLATURE]
 $\sigma(i, j)$: stress intensity at a point (i, j)
 $\epsilon(i, j)$: unit strain at a point (i, j)
 $E(i, j)$: elastic modulus at a point (i, j)

SUBSCRIPT
 c : compressive side or concrete
 cc : concrete in compressive side
 sc : steel bar in compressive side
 t : tensile side
 st : steel bar in tensile side
 n : edge of compressive side

Fig.2 Cross-section of beam or coordinate system for mathematical model of structural mechanism

where σ_n , E_n denote respectively stress intensity and elastic modulus of the materials at the compressive edge, and nomenclature concerning others is denoted in Fig.2.

From Eqs. (7), both the combined stresses "C" and tension "T" are given by:

$$\left. \begin{aligned} C &= \frac{\sigma_n}{nE_n} \left\{ \int_0^n y E_{cc}(i, j) dA_{cc} + (n - d_c) \int_A E_{sc}(i, j) dA_{sc} \right\} \\ T &= \frac{(d - n) \sigma_n}{n E_n} \int_A E_{st}(i, j) dA_{st} \end{aligned} \right\} (8)$$

where y is the distance from the neutral axis to any area element " dA_{cc} ".

On the basis of the stress balance, the combined stresses C and T are equal each other, that is,

$C = T$. Accordingly " n " or the distance from the edge of compressive side to the neutral axis (see Fig.2) can be calculated by Eq.(9)

$$n = \frac{d \int_A E_{st}(i, j) dA_{st} + d_c \int_A E_{sc}(i, j) dA_{sc} - \int_0^n y E_{cc}(i, j) dA_{cc}}{\int_A E_{st}(i, j) dA_{st} + \int_A E_{sc}(i, j) dA_{sc}} \dots \dots \dots (9)$$

As the last term of the numerator of Eq. (9) is a function of n , Eq.(9) can be solved through the following procedure.

If α and β are defined as:

$$\begin{aligned} \alpha &= \int_A E_{st}(i, j) dA_{st} + \int_A E_{sc}(i, j) dA_{sc} \\ \beta &= d \int_A E_{st}(i, j) dA_{st} + d_c \int_A E_{sc}(i, j) dA_{sc} \end{aligned}$$

the following expression $F(n)$ is derived from Eq.(9), and n is obtained as the solution of the equation $F(n)=0$:

$$F(n) = \alpha n + \int_0^n y E_{cc}(i, j) dA_{cc} - \beta \quad \dots\dots (10)$$

$$Y = \frac{\text{summation of moment in compressive side to the neutral axis}}{\text{combined stress of compression}}$$

$$= \frac{\int_0^n y^2 E_{cc}(i, j) dA_{cc} + (n - d_c)^2 \int_A E_{sc}(i, j) dA_{sc}}{\int_0^n y E_{cc}(i, j) dA_{cc} + (n - d_c) \int_A E_{sc}(i, j) dA_{sc}} \quad \dots\dots\dots (11)$$

On the other hand, the geometrical moment of inertia being denoted as I , the bending rigidity EI is given by:

$$EI = \int_0^n y^2 E_{cc}(i, j) dA_{cc} + (n - d_c)^2 \int_A E_{sc}(i, j) dA_{sc} + (d - n)^2 \int_A E_{st}(i, j) dA_{st} \quad \dots\dots\dots (12)$$

And the resistive moments in the cross-section are given by:

$$M_c = C(Y + d - n), \quad M_t = T(y + d - n)$$

where M_c, M_t are the resistive moments standardizing the compressive stress intensity at the edge of compressive side of the concrete and the tensile stress intensity at the reinforcing steel bars in tensile side respectively.

EXPERIMENTAL VERIFICATION OF MATHEMATICAL MODEL FOR STRUCTURAL DAMAGE BY FIRE

In order to verify the validity of the mathematical model for the fire damage estimation, full scale experiments were carried out for two dimensions of reinforced concrete beams

The position where the combined stress of compression acts can be defined by the following Eq.(11) as the distance Y from the neutral axis to the position.

and the results of the experiments were compared with the results of the calculations for the beams specified same as the experimented beams.

Specification of Test Beams

Two different dimensions of beams were tested. The Beam No.1 was 150×300 in cross-section and 1800 in span, and the Beam No.2 was 250×500 in cross-section and 3000 in span (unit: mm).

Mechanical and Thermal Properties of Concrete

Table 2 shows the properties at room temperature of the concrete used for the specimens.

Table 2. Properties of concrete used for test beams

slump (cm)	water/cement ratio : W/C (%)	strength after 4 weeks curing (kg/cm ²)	elastic modulus (t/cm ²)
18	63.9	356	283

Longitudinal section

1. Thermocouples were located 25 points in each cross-section A-A', B-B'.
2. Junction of thermocouples at points 5 and 6 were inserted into reinforcing steel bars.
3. Thermocouples were of butt-welded chromel-alumel and with a thickness of 0.65 mm.
4. Mark X and figure in ○ denote location and number of thermocouple respectively.

Cross section (A-A', B-B')

Fig.3 Location of thermocouple of Beam No. 1

The strength $F(\theta)$ and elastic modulus $E(\theta)$ of the concrete which has been heated to θ in the maximum temperature are given as functions of θ as expressed by Eqs.(13) and (13'), which have

been derived on the basis of the test results on a concrete of river sand and gravel aggregate by T. Harada³⁾

$$\left. \begin{aligned} F(\theta) &= F_0 \\ E(\theta) &= E_0 \end{aligned} \right\} \theta \leq 50^\circ\text{C} \dots\dots\dots (13)$$

$$\left. \begin{aligned} F(\theta) &= F_0 (650 - \theta) / 600, \\ E(\theta) &= E_0 (111 - 0.214 \theta) \times 10^2, \end{aligned} \right\} 50 < \theta \leq 450 (^\circ\text{C}) \dots\dots\dots (13')$$

where F_o and E_o are the strength and elastic modulus of the concrete at room temperature respectively.

The properties of the concrete heated over

$$F(\theta) = 57.4 (\theta / 100)^4 F_o, \quad \theta > 450^\circ\text{C}$$

$$E(\theta) = 15.6 (\theta / 100)^4 E_o, \quad \theta > 500^\circ\text{C}$$

than the upper limit of temperature range shown in Eqs.(13'), which are to be used for the calculation, can be given by Eqs.(13''), which were obtained by means of an approximate extension of Eqs.(13').

..... (13'')

The thermal properties of the concrete which has been heated to θ in the maximum temperature are also given as functions of θ as

$$\lambda(\theta) = 1.69 - 0.00108 \theta$$

$$\kappa(\theta) = (2.75 + 0.83 \theta)^{-1}$$

..... (14)

where

$\lambda(\theta)$: heat conductivity of concrete heated to θ in temperature (Kcal/mh $^\circ\text{C}$)

$\kappa(\theta)$: thermal diffusivity of concrete heated to θ in temperature (m^2/h)

The decreased mechanical properties of concrete which has once heated tend to recover

gradually after cooling. Figs. 4 and 5 show examples of the natural recovery progress on the strength and elasticity of some kinds of concrete tested by T. Harada³⁾.

The moisture content of the Beam No.1 was measured just before the fire resistance test. It was 7% in weight and nearly constant at position from surfaces to center of the Beam.

Fig.4 Natural recovery of strength of concrete exposed to fire (T. Harada)³⁾

Fig.5 Natural recovery of elasticity of concrete exposed to fire (T. Harada)³⁾

Mechanical Properties of Reinforcing Steel Bar

Table 3 shows mechanical properties of the reinforcing bars measured by T. Kaminosono et al, BRI. The measured bars were extracted from the Beam No.1 after the fire resistance test. On the other hand, the thermal properties of the

bars were not provided, because the area ratio of the bars in the cross-section was insignificant in comparison with one of the concrete, accordingly the effect of the steel bars on heat transmission in reinforced concrete beams was regarded to be negligible in the model for such a heat calculation.

Table 3. Mechanical properties of reinforcing steel bars

dimension	fire duration (hr)	yield strength (t/cm ²)	maximum intensity of stress (t/cm ²)	elastic modulus (t/cm ²)	coefficient of extension (%)
D - 13	not exposed	3.45 (100%)	5.15 (100%)	1760 (100%)	24.2 (100%)
	3.0 compress.* tensile*	2.88 (83%)	4.57 (89%)	1651 (94%)	24.4 (101%)
		2.75 (80%)	4.63 (90%)	1710 (97%)	22.8 (94%)
	1.5 compress. tensile	3.49 (101%)	5.25 (102%)	1623 (92%)	22.8 (94%)
		3.89 (98%)	5.07 (98%)	1658 (94%)	23.5 (97%)
	1.0 compress. tensile	3.50 (101%)	5.42 (105%)	1697 (96%)	22.1 (91%)
		3.89 (98%)	5.28 (108%)	1744 (99%)	22.0 (91%)
	6 φ	not exposed	3.40 (100%)	4.30 (100%)	1959 (100%)
3.0		2.46 (72%)	3.35 (78%)	1871 (96%)	33.6 (106%)
1.5		2.76 (81%)	4.34 (101%)	1910 (97%)	29.7 (98%)
1.0		3.18 (94%)	4.62 (107%)	1846 (94%)	29.8 (92%)

note: Mark "*" denotes location of reinforcing bar; compressive or tensile side

Fire Resistance Test

The fire resistance test for the two kinds of beams were carried out on the basis of JIS-A-1304. The temperature-time curve prescribed in the JIS was applied for the test, which is expressed by Eq.(2). Each test specimen located in the test furnace was covered with ALC, that is, autoclaved lightweight concrete slabs on the test beam and over the entire upper opening of the furnace. As for the fire duration, 60, 90 and 180 minutes were adopted for the Beam No.1, and 90, 120, 180 minutes for the Beam No.2. The Beam No.1 heated for 180 minutes was too damaged to test on its mechanical performance. However the

temperature for this specimen could be also measured entirely for the test period, accordingly the comparison of both results of the calculation and the test for temperature was able to be conducted for the full period of the fire test.

Results of Tests and Calculations Temperatures in concrete

The comparison between measured and calculated temperatures in concrete was made only for the Beam No.1. The temperatures were measured at twentyfive thermocouple locations in each cross-section A-A' and B-B' as shown in Fig.3. Fig.6 shows the temperatures measured at seven representative locations, which were

Fig.6 Comparison between measured and calculated temperatures for Beam No. 1

Fig.7 Isothermal line in cross-section (calculated for Beam No. 1)

averaged for the two cross-sections, comparing with temperatures at the locations corresponding to those measured. There is good agreement between the measured and calculated temperatures except one at the location No.22 at 4 cm in depth from the upper surface on the vertical symmetric axis of the cross-section of the beam. The temperature measured at No.22 shows about 100°C higher than the values calculated. The possible reason for such a significant difference at the location is regarded that the beam was significantly heated also from the upper surface of the beam through an aperture between the upper surface of the beam and the lower surface of the ALC floor slab, though the aperture was filled with asbestos before the test. This reason may be clearly proved by the fact that the temperature measured at No.22 should have been lower than one at No.19 if there had been no such an aperture.

From the such results mentioned above, it has been verified that the mathematical model for temperature calculation as one of submodels in a series of the process of estimating the fire damage of structural members may be entirely reliable. Fig.7 shows isothermal lines for the cross-section at the time 60, 120, 180 minutes as examples from the calculated results.

Damage of structural performance

To study the influence of fire exposure on structural performance of the beam, repeated

loading tests were carried out applying concentric loading on two locations along the longitudinal axis for each of the Beam No.1 and No.2 about one month after the fire tests, where shearing span was applied 60cm for No.1, and 80cm for No.2. Furthermore, to obtain criteria for evaluation of the structural damage by fire, the same loading tests were also conducted for nonexposed beams with same specification as exposed to fire. For the Beam No.2, the repeated loading was not applied except for the beam exposed to fire for 180 minutes.

The test results on the relation between yield strength and fire exposure duration are shown in Table 4. On the other hand, the calculated results for resisting moment and initial rigidity are shown in Table 5, comparing with those measured by the structural experiments, where the rigidity was given as an average gradient to the yield point in the load-deflection curve. As this comparison proves clearly, there is satisfactorily good agreement between the theoretical and the experimental results for the structural performance, too.

These results have clarified that the above-described mathematical model composed by two submodels of heat transmission and structural mechanics will enable sufficiently accurate estimation for the fire damage of beams, if the fire conditions such as the fire temperature-time curve are given.

Table 4. Measured yield strength decrease of the beams by fire exposure

fire duration (minute)	0(not exposed)	60	90	120	180
Beam No. 1	8.7 t (100%)	7.8 t (90%)	6.8 t (78%)	—	—
S1 Beam No. 2	45.0 t (100%)	—	41.0 t (91%)	42.0 t (91%)	—
S2	37.0 t (100%)	—	33.5 (91%)	—	30.0 t (81%)

Note: S1 & S2 for Beam No.2 tested in shear span of 80 & 100cm respectively

Table 5. Calculated and measured structural performance of fire-exposed/normal-cured beams; figures in () are measured

specification of beams (mm)	fire duration (min)	resisting moment (t·m)	residual ratio* of strength (%)	rigidity (t·m ²)	residual ratio** of rigidity (%)
Beam No.1 span: 1.8 m	0 [not exposed]	2.2 (2.6)	100 (100)	223 (207)	100 (100)
	30	2.2	98	204	91
	60	2.0 (2.3)	88 (90)	149 (150)	67 (72)
	90	1.8 (2.1)	81 (80)	123 (116)	55 (56)
	120	1.4	65	116	52
	150	1.1	50	114	51
	180	0.7	31	112	50
Beam No.2 span: 3 m	0 [not exposed]	16.3 (18.0)	100 (100)	2730 (2500)	100 (100)
	30	16.1	99	2620	96
	60	15.6	96	2240	82
	90	15.4 (16.5)	95 (92)	2090 (1820)	76 (73)
	120	15.2 (16.8)	94 (93)	1950 (1680)	72 (67)
	150	14.6	90	1810	66
	180	14.1 (15.0)	87 (83)	1680 (1570)	62 (63)

Note: * denotes ratio of resisting moment of fire exposed beam to one of the beam tested before fire exposure.

** denotes ratio of rigidity of fire exposed beam to one of the beam tested before fire exposure.

ESTIMATION OF STRUCTURAL DAMAGE OF VARIOUS KINDS OF BEAMS EXPOSED TO FIRE

To apply the mathematical model for predicting the decrease of structural performance of reinforced concrete beams exposed to fire, calculations were conducted for 720 kinds of beams with specifications listed in Table 6. As for the properties of concrete of the beams, the values shown in Table 2 for the tested beams were also applied for the calculation. The residual structural performance immediately (one day) after the fire experiment was calculated for all the listed beams, and the residual performance in one year after the fire exposure was also calculated only for the beams with a cross-section of 30cm in width and 60cm in depth. Some examples from the results calculated on the ratios of strength and rigidity remaining after the fire exposure to those existing before the fire test are shown in Figs. 8 and 9. These

results clearly illustrate that both the strength and rigidity decrease almost linearly according to the fire exposure duration. Provided the width of a beam is constant, the depth of the beam has no serious effect on the decrease ratio for the rigidity. Fig.8(C) shows an example for the specially specified beams which have width of 50cm and lots of reinforcing steel bars ($P = 3\%$; where P is ratio of the bars to beam in cross-sectional area) in tensile side and without steel bars in the compressive side. It suggests that the fire exposure duration may have serious effect on beams which have few steel bars in compressive side, though they have big cross-section area or heat capacity.

Fig.9 also shows examples of the results calculated for beams with four kinds of dimensions, and $P = 2\%$ and $\gamma = 0.4$ (γ : ratio of bar's cross-sectional area in compressive side to one in tensile side) and the comparison on the structural performance in one day after the fire ex-

posure with one in one year after for the beam $30 \times 60\text{cm}$ in cross-section, to discuss the natural recovery on the structural performance of the fire exposed beams after cooling. It suggests

that the fire damage will be considerably cured as time passed after fire exposure by the natural recovery of mechanical properties of concrete.

Table 6. Specification of beams calculated for fire damage estimation

factors	specifications
width of beam in cross-section: B	B= 20, 30, 40, 50, 60 (cm)
depth of beam in cross-section: D	D= 1.5B, 2B, 2.5B, 3B
ratio of reinforcement: P*	P= 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 (%)
ratio of compressive/tensile reinforcement: γ^*	$\gamma = 0, 0.2, 0.4, 0.6, 0.8, 1.0$

* Note: P= ratio of cross-section area of main reinforcing bars to one of beam
 $\gamma = a_c / a_t$, where a_c , a_t are cross-section area of reinforcing bars in compressive/tensile side respectively.

(a) Beam with B = 20 cm, P = 1.5%, $\gamma = 0.6$

(b) Beam with B = 30 cm, P = 1.5%, $\gamma = 0.6$

(c) Beam with $B = 50$ cm, $P = 3.0\%$, $\gamma = 0.0$

Fig.8 Examples of calculated results of fire damage for various beams

Fig.9 Examples of calculated fire damage of beams with various dimensions

CONCLUSION

The discussion given above shows that the mathematical model for structural damage of fire exposed beams will be sufficiently reliable in accuracy and practical for a wide range of applications in relation to rehabilitation of building structures which have been exposed to fire.

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REFERENCES

- 1) KAWAGOE, K. and SEKINE, T. : Estimation of Fire Temperature-Time Curve in Rooms, BRI Research Paper No.11, 1963
- 2) WAKAMATSU, T. : Heat Flow Analysis of Building Members during Fire Exposure, First and Second Reports, Transactions of the Architectural Institute of Japan No.109 and 111, 1965 (in Japanese)
- 3) HARADA, T. : Fire Endurance Structure, Kogyochosakai (in Japanese)