Mechanical Behaviours of Concrete and Reinforced Concrete at Elevated Temperatures up to 500 °C

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Abstract

The present study reviews and summarizes the test results which have been obtained from the experimental works conducted to investigate physical properties of concrete as well as mechanical behaviours of reinforced concrete (RC) members at elevated tempratures obtained in research organizations including Central Research Institute of Electric Power Industry.

The main purpose of the study was to help work out the rational design approach of RC structures exposed to the above mentioned high temperature conditions occuring in such RC structures as slabs, columns and beams, etc. to be used in LMFBR type nuclear power plant to support reactor vessels or to accommodate high temperature pipes. The present study constitutes a part of the project study, which has been being performed to develop a pool-type LMFBR suitable in Japan.

1 Introduction

Extensive researches for development of pool-type LMFBR has been conducted in Japan to work out a safe and economical nuclear reactor in the future.

CRIEPI's investigation on the feasibility for the development of LMFBR under such severe seismic design conditions as occuring in Japan showed that a pool-type reactor assuring the safe serviceability can be constructed. Following the feasibility study the research activities have been shifted to the rationalization of the design method as a three years' plan beginning in 1984 to achieve safe and economical design.

As a fundamental study related to the above project research, in order to help establish the rational design method of reinforced concrete (RC) members exposed to elevated temperatures, the experiments for physical properties of concrete as well as mechanical behaviours of RC members at elevated temperatures were conducted, in which compressive, tensile splitting and bending strength tests on concrete as well as flexural tests of RC members in the temperature conditions up to 500°C were performed.

In the present paper the lower bound empirical formulas to estimate the temperature dependences of physical properties of concrete were proposed, which may be applied to the design of RC structures at elevated temperatures, based on the experimental results so far obtained in CRIEPI and other research facilities. Also, the influences of high temperatures on mechanical behaviours of RC members were described as compared with those tested at normal temperature. (1) (2)

2 Outline of Experiments

2.1 Strength tests of concrete

Strength tests of concrete were performed in the temperature conditions described in Table 1. Two kinds of concrete were used. One designated as Series (1) has a compressive strength of 400kgf/cm^2 and another as Series (2) had 240kgf/cm^2. Mix proportions of two concretes, the features of aggregates used and the test conditions other than that for temperatures are shown in Table 2, Table 3 and Table 4, respectively.

2.2 Flexural strength tests of RC members

The main properties of RC members tested are listed in Table 5. The schematic diagrams of the heating furnace and the loading apparatus are depicted in Fig. 1. RC specimens were uniformly heated to the fixed temperature conditions by the electric heaters, which were partitioned into twelve parts to facilitate controlling temperatures in the heating furnace. Then, they were flexurally loaded up to failure by the loading apparatus comprised to the H type steel beams and the actuator with the electric servo controller.

Mix proportion of concrete used to RC members was the same as that of Series (1) in the strength tests of concrete. RC specimens were moist cured up to the age of 28 days after casting and then stored under air-dry condition for about 40 to 70 days untill the start of heating. After RC specimens had been heated up to the prescribed temperature at the heating rate of about 100°C/day, the temperature was kept constant for 24 or 48 hours.

Curvatures of RC members at the elastic as well as plastic deformation ranges were measured by means of the linear differential transformer gauges, which were specially manufactured for the measurement at elevated temperatures (Photo. 1), and conventional displacement gauges arranged at the position of the loading point on the H type steel beam, which measured the change of the distance between the two steel beams.

3 Temperature Dependences of Physical Properties of Concrete (3) ~ (26)

3.1 Strength properties

Temperature dependences of concrete strengths were evaluated by the relationships between temperatures and ratios of concrete strengths at elevated temperatures to those under normal temperature, which were obtained from the experiments performed in CRIEPI and other organizations. The results for compressive strength, tensile strength and related ranges of test conditions are shown in Fig. 2, Fig. 3 and Table 6, respectively.

In these figures the data expressed by the solid lines corresponding to the experiments in CRIEPI, showed that concrete strengths at high temperature conditions sustained the values nearly equal to or higher than those at normal temperature in the temperature range up to 400°C and decreased substantially in the range over 400°C and that the strengths at the cooled temperature after high temperature reversals had the tendency to decrease linearly with the increases of the heating period as well as the number of heating and cooling cycles. But, the influences of the conditions of temperature hysteresis on strength properties were not so conspicuous that they could not be quntified.

Judging from the evaluated data, the temperature dependences of compressive and tensile splitting strengths were considered to be almost the same. The decrease in strengths due to temperature rises was relatively small in the temperature range below 200°C and became larger over 200°C.

The temperature dependences of concrete strengths to be applied to the design might be formulated by the bi-linear lines determined as minimum values of the experimental data, which were expressed by the following formulas.

$$R = 0.7 \qquad 100°C \leq T \leq 200°C$$
$$= -0.0018\,T + 1.05 \qquad 200°C \leq T < 600°C$$

where, R; ratio of concrete strength at elevated temperatures to that at normal temperature
T; temperature considered.

3.2 Deformational properties

The temperature dependence of Young's modulus of concrete is given in Fig. 4. The decrease rate of Young's modulus due to temperature rises was considerably large as compared with that of concrete strength. The temperature dependence of Young's modulus based on the plotted data was formulated by the linear relationship expressed below.

$$R = -0.0015\,T + 0.73 \qquad 100°C \leq T < 500°C$$

where, R; ratio of Young's modulus at elevated temperature to that at normal temperature

The relationship between the decrease rate of Young's modulus (G; %) and the loss of weight of concrete specimen (W; %) due to water dispersion and the temperature dependence of compressive strain at maximum compressive stress are shown in Fig. 5 and Fig. 6, respectively.

Linear relationships were found also in these cases. The relationship for the former was expressed by the following formula.

$$W = 0.047G + 2.88 \qquad 100°C \leq T \leq 500°C$$

The compressive strain at maximum compressive stress increased with temperature rises and the value at 500°C was about 2.2 times as large as that at normal temperature. Though the strain properties observed in Fig. 6 do not necessarily indicate the temperature dependence of the compressive strain capacity of concrete, the increasing tendency of which with temperature rises may be conceivable.

4 Mechanical Behaviours of RC Members

4.1 Free deformation behaviours

The influences of the differences of thermal expansion coefficients between concrete and reinforcement on mechanical behaviours, which are considered to be negligible at normal temperature, should be taken into account at elevated temperatures, because the differences of thermal expansion coefficients as well as the value of temperature change become larger.

The relationships between curvatures and temperatures obtained by the free deformation tests of RC members are shown in Fig. 7, in which curvatures occur due to temperature rises in RC members with single reinforcement and increased remarkably around the temperature of about 150°C. These behaviours suggest that the first crack due to the internal restraint stress caused by the differences of thermal expansion coefficients appeared at the temperature of about 150°C and that the number of cracks might increase in the temperature range over 150°C.

The crack pattern observed in H-400D specimen, which was cooled down to normal temperature after heating, is illustrated in Fig. 8. Five major cracks corresponding to one half of the number of cracks after the flexural loading test developed in the test portion of RC members during the heating stage. While the internal stresses at and around the cracked sections were released, the most part of the residual stresses between adjacent cracks was considered to be existent even after the beam was cooled down.

Consequently, it is most probable that the residual internal stress might influence on mechanical behaviours of RC members due to the subsequent loading.

4.2 Strength and deformation capacities of RC members

Table 7 gives the test results of physical properties of concrete and reinforcement at elevated temperatures as well as the strength and the deformational capacities of RC members at yield and ultimate states. It also shows the calculated values of yield and ultimate flexural strength of RC members based on the conventional cracked section thoery and the draft of Limit State Design of RC structures (27), which has been proposed by Japan Society of Civil Engineers, respectively.

Curvatures at the yield state of H-200 and H-300 were measured to be larger as compared with other cases. These results were considered to be attributed to the differences of the yield strains of reinforcements, the release of internal stresses remaining at the section between cracks prior to loading, the deterioration of bond strength, the fine cracks on the compression fiber of RC members due to preceeding elevated temperatures, etc.. The ultimate deformational capacities of RC members at elevated temperatures were not so much different as that at normal temperature and also the differences of those between RC members at elevated temperatures were not found. However, those at normal temperature after heating and cooling cycle , that is, NH-200 and NH-500 were small.

The yield strengths of RC members at elevated temperatures obtained from the experiments tended to decrease with temperature rises and the correspondences of those obtained from the experiments and those calculated by the conventional cracked section theory were fairly good.

The ultimate strength of H-300 was the largest among all the RC specimens and became smaller in the order of N-B, H-200, H-100, H-400, and H-500. The order of the strengths of RC beams corresponded to that of the tensile strengths of reinforcements in the deformation range of about 5% at the temperatures at which the beams were tested. It was suggested that the ultimate strength capacities of RC members at elevated temperatures were considered to be mainly governed by the stress-strain relationship of reinforcement in the large deformation range at elevated temperatures, being hardly influenced by the concrete properties.

The calculated values of the ultimate strength capacities, based on the draft of Limit State Design of RC structures as well as the assumption of the elasto-plastic relationship between stress and strain of reinforcement, were considerably lower than the experimental values.

5 Conclusions

Conclusions derived within the limit of the experimental study were summarized as follows.

1) The formulas to estimate strength and elastic properties of concrete at elevated temperatures to be applied to the practical design were suggested based on the experimental results so far obtained.
2) The internal stresses due to the differences of thermal expansion coefficients between concrete and reinforcement increase due to temperature rises and may cause cracking in RC members under free deformation without external restraint. The influences of the internal stresses remaining at the sections between cracks prior to loading on mechanical behaviours of the beams during loading stage may be considerably large.
3) Yield strengths of RC members at elevated temperatures were able to be estimated by the conventional cracked section theory, taking into account the temperature dependences of mechanical properties of concrete and reinforcement. The calculations of the ultimate strength capacities of RC members based on the draft of Limit State Design of RC structures as well as the assumption of the elasto-plastic relationship between stress and strain in reinforcement led to a marked conservative results.

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Table I Temperature conditions for concrete strength test

Concrete	Test condition	Sustained or repeated	Temperature (℃) 100	200	300	400	500
Series (1)	At high temp.	5 hours	○	○	○	○	○
	At normal temp. after a fixed elapsed time of sustained temp.	10 days	○	○	○	○	○
		30 days	○	○	○	○	○
	At normal temp. after the number of cycles of temp. reversal between 100℃ and a fixed high temp.	1 cycle	○	○	○	○	○
		10 cycles	—	○	○	○	○
		30 cycles	—	○	○	○	○
Series (2)	At normal temp. after the number of cycles of temp. reversal between 100℃ and a fixed high temp.	15 cycles	○	○	○	○	○

Table II Mix proportions of concrete

Concrete Series	Max. size of aggregate (mm)	Slump (cm)	Air (%)	W/C (%)	S/A (%)	Unit Content (kg/m³) W	C	FA	CA	WRA
(1)	20	8±2.5	3±1	43.2	43.1	160	370	789	1103	0.925
(2)	20	10±2.5	3±1	64.8	47.0	162	250	908	1082	0.625

W ; water C ; concrete S ; sand A ; aggregate FA ; fine aggregate
CA ; coarse aggregate WRA ; water reducing agency

Table III Features of aggregates

Kinds of aggregate	Mineralogical classification	Specific gravity under saturated surface dry	Absorption (%)	Finess modulus
Sand	Chart, Sandstone Quartz, Granite Andesite, Slate	2.64	0.99	2.58
Crushed stone 5~13mm	Sandstone Quartz, Orthoclase	2.71	0.70	---
13~20mm		2.80	0.62	---
Mix of crushed stone	Diabasic tuff Quartz, Chlorite	2.80	0.66	6.74

Table IV Details of concrete strength tests

Curing before heating ; moisture during heating ; unsealed	Age at the start of heating ; 28~35 days
Rate of heatig ; below 1.0℃/min cooling ; below 0.2℃/min	Temp. condition ; high at test ; normal
Size of test specimen comp. ; φ10cm×20cm tens. ; φ15cm×20cm bend. ; 10cm×10cm×40cm	Duration after cooling till test ; 1 day

Table V Details of RC members

RC specimen	As (cm²)	Temp. at test (℃)	Temp. at heating (℃)
N−D	3.972	normal	100
NH−200	3.972	normal	200
NH−500	3.972	normal	500
H−100	3.097	100	100
H−200	3.163	200	200
H−300	3.972	300	300
H−400	3.972	400	400
H−400D	3.972	400	400
H−500	3.972	500	500

Test length ; 180cm
Heating length ; 220cm
Cross section ; 20cm×40cm
Effective depth ; 35cm
Diameter of reinf. ; 16mm
Reinf. ratio ; about 0.57%
As ; total cross sectional area of tensile reinf.
H−400D ; cross section with double reinf.

Table VI Test conditions of the results so far obtained

Kinds of cement	Aggregate	W/C
• normal • aluminous • blast−furnace slag	• serpentine • expanded shale • lime stone • sandstone • basalt • blast−furnace slag • andesite • light−weight • river gravel	• 26.4~70.0%
Unit content of cement	Holding time of elevated temperatures	Curing period before heating
• 300kg/m³~ 450kg/m³	• 20 minutes • 1 hour • 1 week • 4 weeks • 13 weeks	• 4 weeks~ 1 year
Rate of heating	Rate of cooling	Temp. condition at test
• 10℃/hr. • 60℃/hr.	• 90℃/hr. • 10℃/hr. • 120℃/hr.	• normal temp. • high temp.

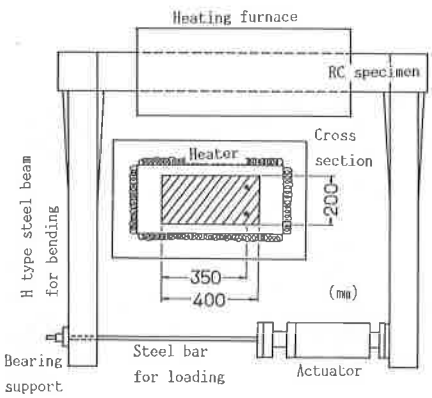

Fig. 1 Heating furnace and loading apparatus

Photo. 1 Displacement gauge for high temperatures

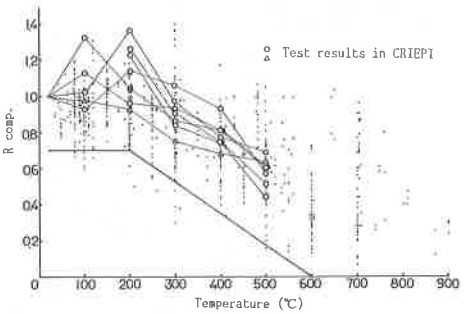

Fig. 2 Temperature dependence of compressive strength of concrete at elevated temperatures

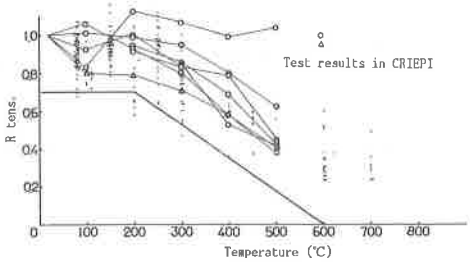

Fig. 3 Temperature dependence of tensile splitting strength of concrete at elevated temperatures

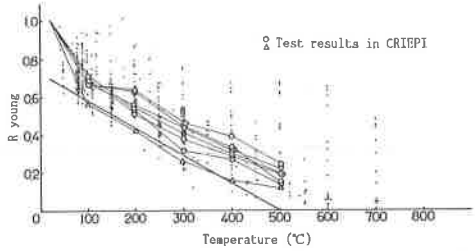

Fig. 4 Temperature dependence of Young's modulus of concrete at elevated temperatures

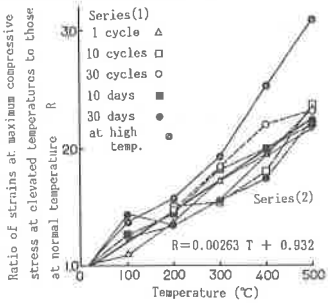

Fig. 6 Temperature dependence of compressive strain at maximum stress at elevated temperatures

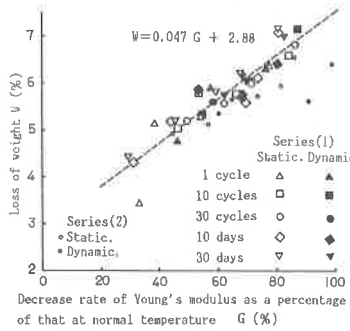

Fig. 5 Relationships between decrease rates of Young's modulus and loss of weight of concrete specimens

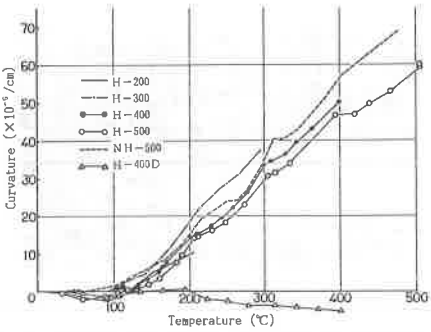

Fig. 7 Relationships between temperature and curvature under free deformation tests of RC members

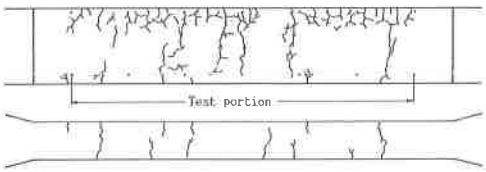

Fig. 8 Crack pattern of RC member at normal temperature after heating and cooling cycle

Table VI Test results of RC members performed in CRIEPI

Test Specimen	σ cu (kgf/cm²)	Ec ×10⁵ (kgf/cm²)	σ bu (kgf/cm²)	σ tu (kgf/cm²)	Es ×10⁶ (kgf/cm²)	σ sy (kgf/cm²)	My Exp. (t−m)	My Cal. (t−m)	Mu Exp. (t−m)	Mu Cal. (t−m)	φ y ×10⁻⁶ (cm⁻¹)	φ u ×10⁻⁶ (cm⁻¹)
N−B	516	3.48	62.7	39.0	2.07	4000	5.87	5.15	7.00	5.42	73	1049
NH-200	450*	1.97*	50.0*	32.6*	2.07	4000	5.87	5.02	6.58	5.40	78	866
NH-500	250*	0.64*	18.4*	13.0*	2.07	4000	4.64	4.73	5.39	5.28	99	737
H-100	470*	2.34*	56.0*	34.3*	2.06*	3950	4.10	3.94	6.13	4.19	71	1133
H-200	490*	2.03*	50.0*	35.0*	2.01*	3850	4.35	3.90	6.57	4.17	94	1058
H-300	450*	1.88*	34.3*	26.1*	1.93*	2900	4.40	3.65	7.09	3.94	84	992
H-400	363	1.68	49.2	27.7	1.81*	2700	3.50	3.39	5.73	3.67	67	1032
H-400D	335*	1.10*	33.1*	22.3*	1.81*	2700	2.90	3.33	6.01	3.89	81	1005
H-500	220	0.79	27.5	21.7	1.66*	2400	2.75	2.91	3.52	3.22	66	1017

* ; Predicted values based on the experimental results so far obtained

σ cu ; Compressive strength Ec ; Young's modulus of concrete σ bu ; modulus of rupture

σ tu ; Tensile splitting strength Es ; Young's modulus of reinf. σ sy ; Yield strength of reinf.

My, Mu ; Yield and ultimate strengths of RC members, respectively

φ y, φ u ; Curvatures at yield and ultimate states, respectively