Strength Development of High-Strength Concrete in Structure

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Abstract

Because of the high unit cement content in the concrete mix, major concrete temperature rises are observed in the initial stages of hardening in structural members with large cross-sections made of high-strength concrete. While this temperature rise in the initial stages of hardening contributes to the initial development of the concrete strength, it also causes thermal cracking and obstructs medium to long-term increases of the concrete strength.

In the study reports below, investigations were made on the effects of the concrete temperature rise in the initial stages of hardening on the medium to long-term development of the strength of structural concrete between the ages of 28 and 91 days. In the study, comparisons were made, for example, between the compressive strength of a control specimen subjected to standard curing at 28 days and the compressive strength of core specimens taken from structural members, and observations were made on the methods of evaluating the concrete strength in structure, defined here as the compressive strength of core specimens at 91 days.

The results obtained indicate that, when the maximum temperature of the concrete is the structure does not exceed 60 ℃, the concrete strength in structure at the age of long-term will generally be greater than the compressive strength of the standard-curing specimens at 28 days, allowing one to evaluate the strength of the structural concrete in terms of the compressive strength of the 28-days standard-curing specimens. When, on the other hand, the maximum temperature of the concrete in the structure exceeds 60 ℃, the strength in concrete structure may be smaller than the compressive strength of the 28-days standard-curing specimens, creating risks in the evaluation of the concrete strength in structure by the latter.

1. Introduction

In order to ensure the structural safety of a reinforced concrete building structure, it is necessary that the concrete in the structure attain the required strength. Measurement of the compressive strength of cores drilled from the structure is regarded as the best way to assess the strength of concrete in the structure, but cores cannot always be drilled. For this reason, control specimens made of concrete sampled on the job site and water-cured on site or standard-cured are normally used to evaluate the strength of concrete in the structure after grasping the relationship between the strengths of drilled cores and control specimens. The Japanese Architectural Standard Specification for Reinforced Concrete Work (JASS 5) assumes that the strengths of concrete in the structure of 36 N/mm^2 or less are assessable using specimens water-cured on site and provides a proportioning strength equation based on the difference in the strength development between specimens water-cured on site and those standard-cured.

For high strength concrete with a design strength of over 36 mm, however, the relationship between the strengths of drilled cores and control specimens has not been fully elucidated and is therefore currently determined by fabricating full-scale member models for each project to select the proportioning strength and strength-controlling methods.

In this study, the authors collected compressive strength data of cores drilled from high strength concrete full-scale models to elucidate the strength-developing properties and strength distribution of high strength concrete structures and their relationships with the strength of control specimens, thereby investigating the methods of establishing the proportioning strength and control method.

2. Overview of data and construction experiments under analysis

2.1 Data under analysis

A huge amount of data in regard to strength-developing properties of high strength concrete structures have been made available as a construction experiment report submitted to the Technical Instruction Committee for High-rise Reinforced Concrete Structures in the Building Center of Japan. This technical instruction committee has provided technical instructions to 51 cases, in which 46 cases applied for committee instruction in 1986 and later. The reports of 41 cases among them having clear indications of the locations of cores from the structures, compressive strength, proportioning conditions, etc., were adopted as data for analysis. The report of construction experiments of full-scale models resulting from the Ministry of Construction's R&D Project for Superlightweight Reinforced Concrete Skyscrapers was also used for the analysis.

The maximum design strength for concrete in each case submitted to the Technical Instruction Committee of the Building Center of Japan ranged from 36 to 60 N/mm^2. The report of the MOC's R&D project included design strengths of 60 and 100 N/mm^2, but only the cases of 60 N/mm^2 are investigated in this study.

Most of these reports excepting a few use a conventional unit (kgf/cm^2) for concrete strength, but the values were analyzed in an SI unit in this study by assuming 10 kgf/cm^2 = 1 N/mm^2.

2.2 Construction method overview

Vertical (column) and horizontal (beam/slab) members may either be placed separately in two days or monolithically in one day. The cases under analysis mostly adopted separate placing, with a few exceptions for monolithic placing. The slump requirements are mostly 18 cm, excepting some of the cases with 21 cm when the design strength is 60 N/mm^2 or 48 N/mm^2. Normal portland cement was used as the cement in all cases, though the use of low-heat portland cement with a high fineness has been increasing in recent years. As for chemical admixtures, superplasticizers were used up to 1987 and air-entraining and high-range water-reducing admixtures were used thereafter.

Reinforcement was preassembled. The forms were mostly system forms. The normal procedure of separate placing of vertical and horizontal members was as follows: preassembling of column reinforcement – erection of column reinforcement and joint work – formwork construction for columns – placing of column concrete – formwork construction for slab/beams – arranging of preassembled beam reinforcement and placing of slab reinforcement – placing of beam and slab concrete. In some cases, formwork construction for slab/beams was done before placing of column concrete.

2.3 Construction experiment overview

The experiments described in the construction experiment report submitted to the

Technical Instruction Committee consisted of execution-verifying experiments (also referred to as full-scale model construction experiments) and preliminary experiments for construction. Execution-verifying experiments are intended to confirm the strength of concrete in the structure, dimensional accuracy, finishing, workability, and other factors by constructing a frame consisting of columns, beams, and slabs representing one to three spans of one or two floors in the lower layers, where the design strength of the trial design is highest, and executing the design according to the procedure mentioned in the previous section. Preliminary experiments for construction include various experiments, such as those nearly the same as execution-verifying experiments carried out for rehearsal and trial placing in a single column member.

Though the construction experiments covered beam and slab concrete as well, this analysis dealt with only column concrete in regard to the strength properties of concrete in the structure. Also, this paper only covers the case in which high strength concrete produced at a ready-mixed concrete plant was placed outdoors in full-scale column members. Partial models simulating the thermal conditions by being covered with insulation were excluded from this study. Full-scale models made with and without reinforcement were both included in this study.

The data of the construction experiments under analysis are tabulated in Table 1. The number of experiments refers to the number of placing days. The proportioning strengths and water-cement ratios are the values at the last execution-verifying experiment of each project. The overall number of experiments is 120, producing 322 column members in total from which cores were drilled to measure the compressive strength. The compression test ages of cores were mostly 28 days in the 1980s, whereas in the 1990s tests were conducted at 91 days as well in most cases. The number of cores drilled from a member for testing at an age ranged from 2 to 44 and was 12.5 on average.

3. Analysis results and discussion

3.1 Effect of core directions

The directions of drilling cores from members in construction experiments were longitudinal and/or transverse. Data of cases where multiple cores were taken both in longitudinal and transverse directions from the same member at the same age were collected to examine the effect of coring directions on their compressive strengths. The relationship between their averages is shown in Figure1. Forty-three members produced by 12 companies were available as examples of cores in both drilling directions from the same member. Figure 1 reveals that no appreciable differences in strength were observed between the directions, though longitudinal cores tended to exhibit slightly higher strength. Accordingly, the directions of cores are ignored in the subsequent discussion.

3.2 Scatter of compressive strength within structures

Figure 2 shows the relationship between the number of cores drilled from a member and the standard deviation of the compressive strengths of such cores. This figure reveals no relationship between the number of cores and the standard deviation of their compressive strengths.

In this light, the relationship between the average and the standard deviation of compressive strengths within the same members was examined as shown in Figure3, selecting members from which 3 or more cores were drilled. This figure reveals that, in the compressive strength range of 36 to 80 N/mm^2, the standard deviations range between 1 and 8 N/mm^2 almost independently of the compressive strength. These standard deviations expressed by coefficients of variation are 6 to 7% on average and 11 to 12% at the maximum.

3.3 Strength development of concrete in structures

Figure 4 shows the relationship between the average 28-day strengths and the average 91-day strengths of cores taken from the same members. It is found from the figure that the compressive strength of concrete in structures at 91 days is approximately 1.12 times that at 28 days.

Figure 5 then shows the effect of the maximum temperatures of members on their ratios of 91-day strength to 28-day strength. This figure reveals that the ratio of 91-day strength to 28-day strength decreases as the maximum temperature of concrete increases and that a maximum temperature of 75°C can cause zero-increases over the 28 day-strengths at a probability of 20%.

3.4 Relationship between compressive strengths of control specimens and core specimens

In the case of normal strength concrete, it is required that the in-place strength be estimated by the compressive strength of specimens water-cured on site or seal-cured on site. For high strength concrete under analysis as well, the relationship between the 28-day strength of specimens water-cured on site and the 28- and 91-day strengths of cores was examined as shown in Fig. 6. Figure 7 shows the relationship between the 91-day strength of specimens seal-cured on site and the 91-day strength of cores. The ratio of core strength to 28-day strength of specimens water-cured on site is 93% on average at 28 days but increases to 102% on average at 91 days. The ratio of 91-day strengths of cores to specimens seal-cured on site is 97% on average. These relationships appear to be similar to those of normal strength concrete.

Figure 8 shows the strength differences between standard-cured specimens and water-cured specimens related to mean air temperatures. Figure 9 shows the case of the strength differences between standard-cured specimens and seal-cured specimens. Since no temperature data on site were available, those for Tokyo, Nagoya, and Osaka were applied according to the Chronological Scientific Tables compiled by the National Astronomical Observatory. These figures reveal that high strength concrete with a design strength of more than 36 N/mm^2 exhibits similar values on average to the strength correction values based on the estimated mean air temperature specified in JASS 5, but the scatters are significantly wider. Most columns examined in this study were mass concrete having a cross-sectional area of 80 by 80 cm or 100 by 100 cm. It is therefore more appropriate to specify correction values for strength based on the mean curing temperature (concrete temperature) than on the mean air temperature.

Due to its high cement content, high strength concrete in structures is subjected to high temperature histories at early ages, which are different from those of specimens water-cured on site or seal-cured on site. It is therefore irrational to evaluate the in-place strength by the strength of specimens water- or seal-cured on site.

Accordingly, it was decided to evaluate the in-place strength based on the compressive strength of standard-cured specimens, the so-called potential strength of concrete. The relationship between the core strength and the 28-day strength of standard-cured specimens is as shown in Fig. 10. At 28 days, the core strength is approximately 90% on average of the 28-day strength of standard-cured specimens, but increases to around 99% on average at 91 days, being nearly equal to the 28-day strength of standard-cured specimens.

The relationship between the maximum temperature and the ratio of 91-day core strength to 28-day standard-cured specimen strength is shown in Fig. 11. This figure reveals that the ratio of 91-day core strength to 28-day standard-cured specimen strength decreases by 0.0036 as the maximum concrete temperature increases by 1 degree Celsius. It also shows that the 28-day compressive strength of standard-cured specimens, or the potential strength, may not be achieved by nearly 50% of the concretes when the maximum temperature is as high as 60°C. Accordingly, it is considered more appropriate to increase the proportioning strength by the following equation, as the ratio of decrease in the strength is 0.0036 per degree Celsius when the maximum concrete temperature exceeds 60°C.

$$\alpha_T = 1 + 0.0036 \times (T_{\max} - 60) \qquad \cdots\cdots\cdots \ (1)$$

where α_T = extra factor for proportioning strength

T_{max} = maximum temperature of concrete (°C)

The discussion above is summarized as follows: when the strength of concrete in structures is defined as the compressive strength of cores drilled at an age of 91 days, it can be regarded as the compressive strength of standard-cured specimens at an age of 28 days, or the potential strength, if the maximum concrete temperature is 60°C or less. If the maximum temperature exceeds 60°C, then the in-place strength can be regarded as the value of 28-day strength of standard-cured specimens divided by the extra factor given in Eq. (1).

3.5 Temperature rise of concrete in structures

Long-term strength of concrete is known to remain low when the concrete is subjected to a high temperature history at early ages. Figure 12 shows the placing temperatures or as-delivered temperatures and the maximum temperatures of members in each month. The figure reveals that a higher placing temperature or as-delivered temperature tends to lead to a higher temperature rise. Placing temperature is therefore a possible factor affecting the temperature rise of concrete in structures along with the cement content and member size. Therefore a multiple regression was carried out using these factors and maximum temperature of concrete in structures as the explanatory variables and dependent variable, respectively, as given in Table 2. The maximum temperature of concrete in structures is expressed by Eq. (2). The estimations by Eq. (2) agree well with the measurements as shown in Fig. 13.

$$T_{max} = 0.058 \bullet C + 0.043 \bullet D + 1.61 \bullet T_0 - 44.2 \qquad \cdots\cdots\cdots \ (2)$$

where T_{max} = maximum temperature of concrete (°C)

C = cement content (kg/m^3)

D = column size (mm)

T_0 = placing temperature of concrete (°C)

3.6 Variation of 28-day compressive strength of standard-cured specimens

In the construction experiments examined in this study, concrete samples were taken from 2 to 4 agitating trucks for placing columns. For placing beams and slabs, 3 to 10 trucks were sampled. Samples from multiple trucks were also used when placing slabs on grade. Intraday variations, day-to-day variations, and total variations were therefore examined using these data. The 28-day compressive strengths of specimens made of concrete from one agitating truck were averaged to give a single figure, and an intraday variation was determined from the standard deviation, σ_a, of such figures from two to four agitating trucks. Similarly, the compressive strengths of column concrete or beam and slab concrete samples in a placing day were averaged as a single figure, and a day-to-day variation was determined from their standard deviation, σ_b. When a slab on grade was placed, the compressive strengths of samples from the concrete were also averaged as a single figure for calculating σ_b. Figure 14 shows the relationship between the intraday standard deviation and the day-to-day standard deviation, but no appreciable correlation is observed between them. Figure 15 shows the relationship between the average compressive strength and the standard deviation. The intraday standard deviation was 3.8% on average and 7.2% at the maximum of the average compressive strength, whereas the day-to-day variation was 4.7% on average and 9.8% at the maximum.

The total variation, σ_T^2, was determined as the sum of the intraday variation, σ_a^2, and day-to-day variation, σ_b^2. The total standard deviation was 6.2% on average and 10.2% at the maximum. These values agree relatively well with the results of past research.

Consequently, it is considered appropriate to assume that the standard deviation (total variation) of 28-day compressive strength of standard-cured high strength concrete specimens is not more than 10% of the average compressive strength.

4. Conclusions

Compressive strength data of cores drilled from full-scale models of high strength concrete made using normal portland cement with a design strength of 36 to 60 N/mm^2 were collected to examine their strength-developing properties and the method of determining the proportioning strength. The results are summarized as follows:

(1) Longitudinal and transverse drilling of cores cause no appreciable differences between their compressive strengths, though longitudinal cores exhibit slightly higher strength.

(2) The standard deviation of the compressive strengths of cores drilled from a single member ranges between 1 and 8 N/mm^2, with a coefficient of variation of 6 to 7% on average and 11 to 12% at the maximum.

(3) The in-place compressive strength at 91 days is 1.2 times the value at 28 days on average, but the ratio decreases as the maximum temperature increases. When the maximum temperature is 75°C, 20% of concretes exhibit no strength gains over the period.

(4) The ratio of the 91-day core strength to the 28-day strength of standard-cured specimens decreases to below 1 at a probability of 50% when the maximum concrete temperature reaches 60°C. Accordingly, an extra of $1 + 0.0036\ (T_{max} - 60)$ (where T_{max} = maximum concrete temperature) should be added to the proportioning strength where the maximum concrete temperature is expected to exceed 60°C.

(5) The temperature rise of concrete in structures can be expressed well with Eq. (2) as a function of the cement content, member size, and placing temperature.

(6) The standard deviation of 28-day strengths of standard-cured specimens is

approximately not more than 10% of the average strength.

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Table 1 Overview of high strength concrete full scale columun construction experiment

No.	Period	Place	No. of ex.	Menber size (mm)	No. of columuns	Design stre. (N/mm^2)	Proportioning stre.(N/mm^2)	W/C (%)	Cement content(kg/m^3)	Placing Temp.(℃)	Max. temp. (℃)	No.of specimen	
												28d	91d
1	1986.4～1986.5	Aichi / Kanagawa	2	850,900	4	42.0	47.8	39.0	395～421	～25.0	56.0	72	54
2	1986.1～1986.9	Chiba	2	850	4	48.0	64.0	34.5	481	8.0～28.0	78.0	51	18
3	1986.9	Kanagawa	2	850,950	4	36.0	44.0	40.0	438	27.5～29.0	unknown	24	0
4	1986.10～1987.2	Kanagawa	2	900	4	36.0	49.0	38.0	408～448	15.0～22.0	48.0	48	0
5	1987.2	Saitama	1	900	3	42.0	53.5	34.0	515	–	unknown	54	54
6	1987.1～1987.9	Chiba	2	900,950	6	36.0	50.0	40.0	405～413	10.0～33.0	63.0	84	0
7	1986.4～1987.10	Tokyo	3	800	11	36.0	48.8	38.0	377～468	17.5～31.0	68.0	70	0
8	1987.4～1987.12	Kanagawa	2	900	8	42.0	51.0	35.0	459～486	17.0～24.0	64.0	56	36
9	1987.11～1988.3	Ibaraki	2	900	8	36.0	46.8	36.0	473～486	14.5～20.0	45.0	99	0
10	1987.12～1988.4	Kanagawa	2	1000	3	42.0	-	33.0	515	18.5～26.0	66.7	26	0
11	1988.8～1988.11	Saitama	2	850,900	5	42.0	53.5	35.0	447～475	17.0～32.0	40.0	100	0
12	1988.12～1989.9	Ibaraki	2	900	3	36.0	43.0	44.7	350～369	13.0～32.0	59.7	54	9
13	1989.9	Chiba	1	900	3	42.0	-	36.0	464	31.5～32.0	61.2	72	0
14	1989.9	Osaka	1	1000	2	42.0	-	40.0	438	31.0～32.5	65.5	24	0
15	1988.11～1990.2	Saitama	5	800～900	14	42.0	55.8	35.5	367～500	15.5～22.0	65.2	89	0
16	1989.9～1990.7	Hyogo	4	1000	8	42.0	51.8	40.0	425～447	11.0～33.0	72.0	66	20
17	1988.9～1991.3	Chiba	4	850,900	16	42.0	60.0	30.5	477～548	15.0～32.5	66.0	118	104
18	1990.7～1991.3	Chiba	2	950	6	42.0	-	33.0	457～485	16.0～36.0	78.0	117	0
19	1989.12～1991.3	Saitama	4	850～950	9	36.0	49.0	38.0	378～447	16.0～33.0	59.7	112	61
20	1989.12～1991.3	Ibaraki	6	850,950	18	42.0	50.4	38.0	354～459	10.5～35.0	86.3	224	76
21	1991.10～1991.11	Ibaraki	3	850	10	60.0	-	27.0	611	24.0～27.0	76.0	152	116
22	1991.12	Tochigi	1	900	1	48.0	64.5	32.8	439～517	18.4～26.3	63.0	10	28
23	1991.1～1992.1	Osaka	4	950,970	15	42.0	58.2	35.0	398～500	13.0～34.0	58.0	176	0
24	1991.8～1992.3	Kanagawa	3	900,1000	5	42.0	-	39.5	413～446	17.0～27.5	unknown	66	75
25	1991.9～1992.11	Chiba	5	650～750	11	54.0	66.5	29.0	425～630	19.0～34.0	80.0	178	33
26	1993.3	Tokyo	1	850	4	42.0	57.0	36.0	459	19.7～20.1	53.0	72	72
27	1992.6～1993.6	Chiba	2	850,900	8	42.0	50.4	41.0	415	28.2～29.4	unknown	88	48
28	1991.9～1993.8	Saitama	5	900	15	42.0	55.4	36.0	425～472	17.5～35.5	72.0	181	92
29	1992.1～1993.8	Chiba	3	900,950	11	48.0	56.8	34.0	446～416	16.0～31.0	73.0	54	74
30	1993.2～1994.2	Kanagawa	6	900,950	22	42.0	63.4	35.5	314～532	11.5～33.6	88.5	326	322
31	1993.10～1994.3	Kanagawa	2	850	3	48.0	63.0	30.0	413～500	18.0～	63.0	36	36
32	1993.11～1994.10	Chiba	2	900,950	3	42.0	56.0	39.0	415～486	17.0～23.0	62.1	46	45
33	1994.6～1994.11	Chiba	2	850,950	8	42.0	50.4	40.5	407～415	19.0～	59.6	113	89
34	1994.11	Chiba	1	900	2	42.0	55.8	35.5	451	16.0	41.5	17	18
35	1993.12～1994.12	Saitama	2	950	5	42.0	50.4	39.0	423	14.5～15.2	50.6	35	44
36	1993.9～1995.7	Chiba	6	900,950	15	42.0	62.4	36.0	378～472	13.8～34.7	78.3	363	362
37	1994.8～1995.10	Ibaraki	5	900,950	10	48.0	69.4	36.6	447～531	11.0～37.0	89.2	162	331
38	1995.10	Tokyo	1	900	1	42.0	56.0	39.5	418	29.1	64.4	0	21
39	1990.8～1995.11	Osaka / Tokyo	5	900,950	12	42.0	57.6	31.0	389～545	13.0～35.0	83.0	90	72
40	1994.10～1996.5	Hyogo	3	850～1000	11	42.0	61.8	36.0	380～565	12.1～26.8	80.1	142	141
41	1997.4～1998.1	Osaka	3	900	6	60.0	83.0	33.0	531～550	13.3～31.0	81.0	23	35
42	1998.4～1999.3	Chiba	2	950	5	48.0	61.7	34.5	493～567	19.0～24.5	93.5	57	75

Table 2 Factors affecting temperature rise

Explanantory variavble	Partial regression coefficient	Standard error	t value
Intercept	-44.2	12.94	-3.42
C	0.058	0.0082	7.03
D	0.043	0.0119	3.64
T_0	1.61	0.920	17.46

Multiple correlation coefficient (R)	:0.845
Contribution (R^2)	:0.713
Contribution adjusted for degree of freedom	:0.708
Standard error	:8.193
No. of observations	:148

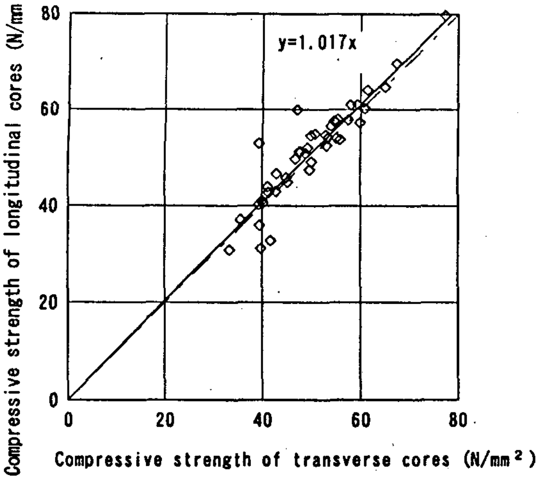

Fig.1 Effect of core direction

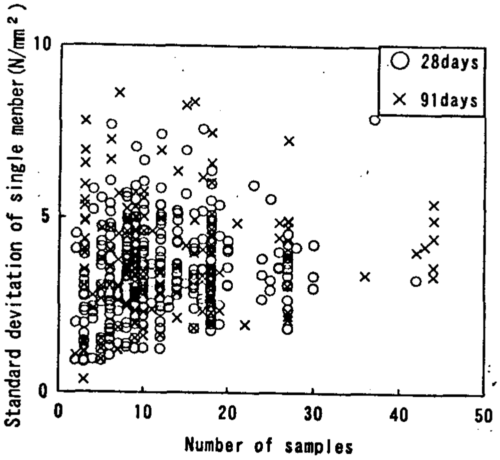

Fig.2 Relationship between number of cores and standard devitation

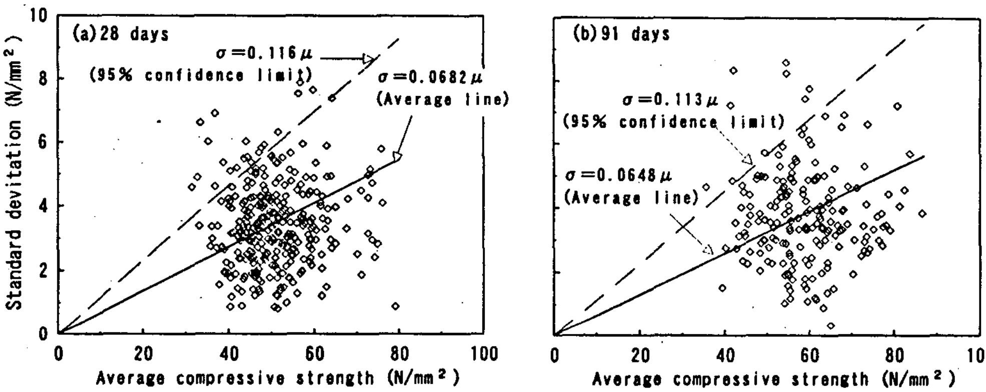

Fig.3 Averages and standard devitations of in-place strength

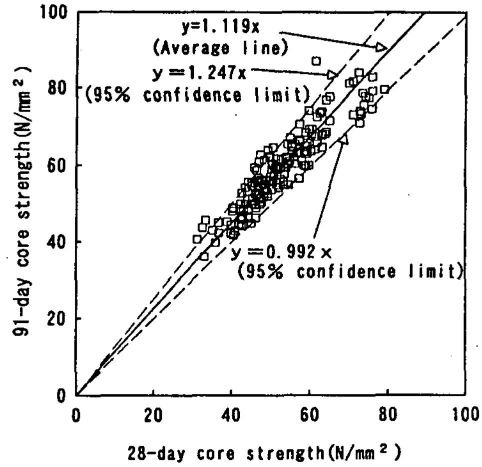

Fig.4 Relationship between average 28-day and 91-day compressive strength of cores drilled from the same member

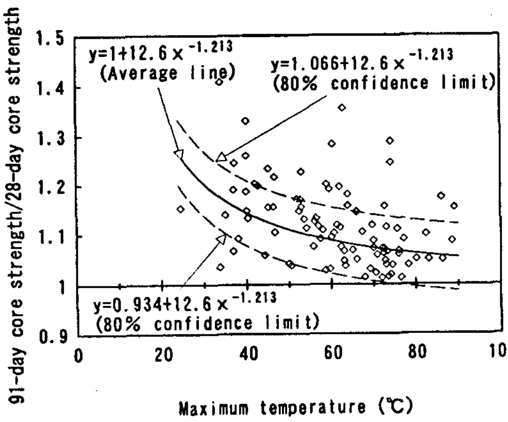

Fig.5 Effect of maximum concrete temperature on core strength

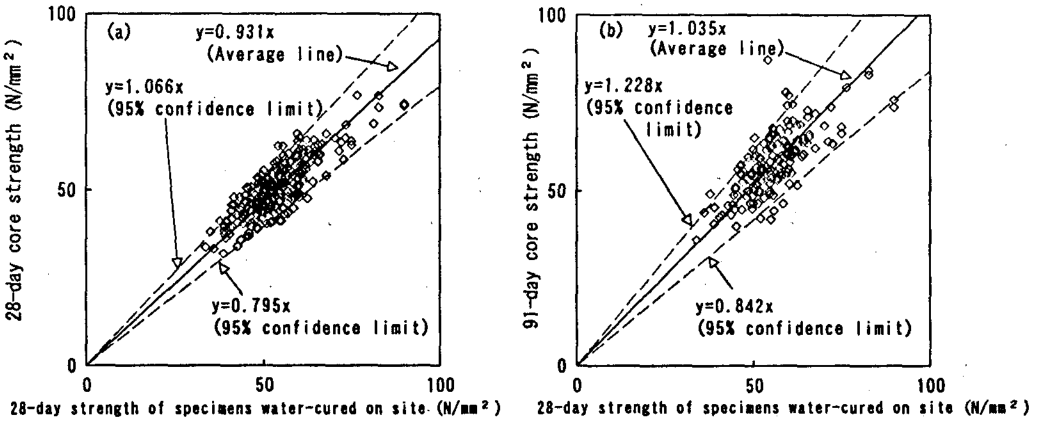

Fig.6 Relationship between compressive strength of specimens water-cured on site and drilled cores

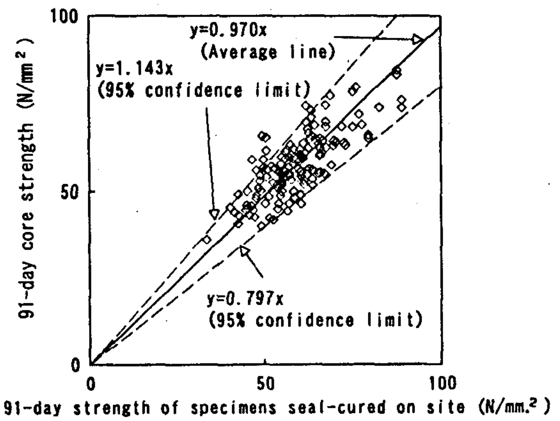

Fig.7 Relationship between compressive strength of specimens seal-cured on site and drilled cores

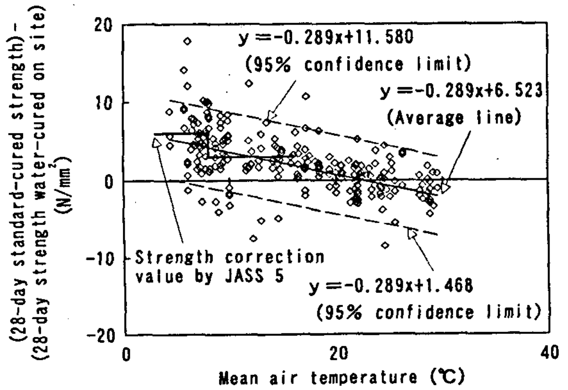

Fig.8 Relationship between mean air temperature and strength difference

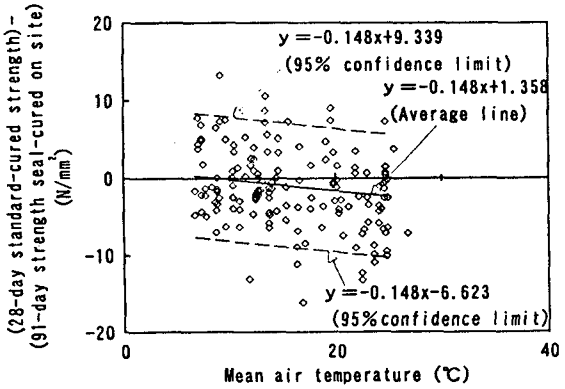

Fig.9 Relationship between mean air temperature and strength difference

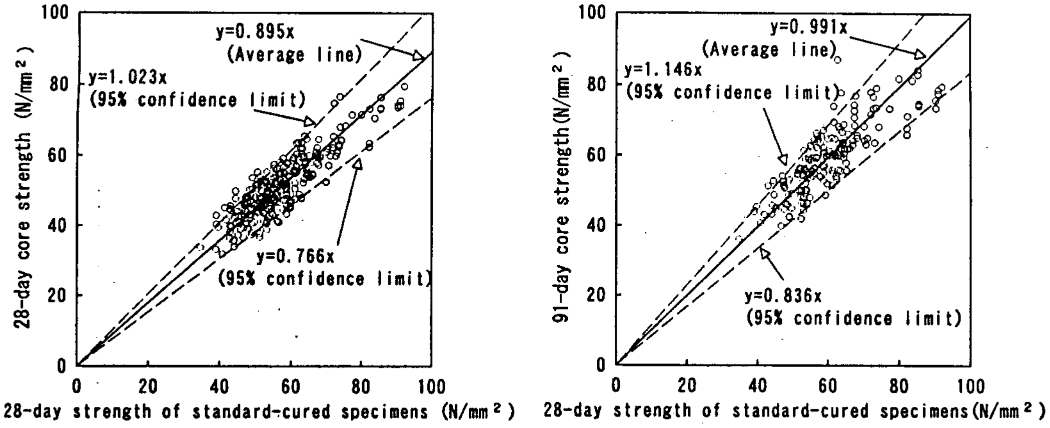

Fig.10 28-day and 91-day core strength related to 28-day strength of standard-cured specimens

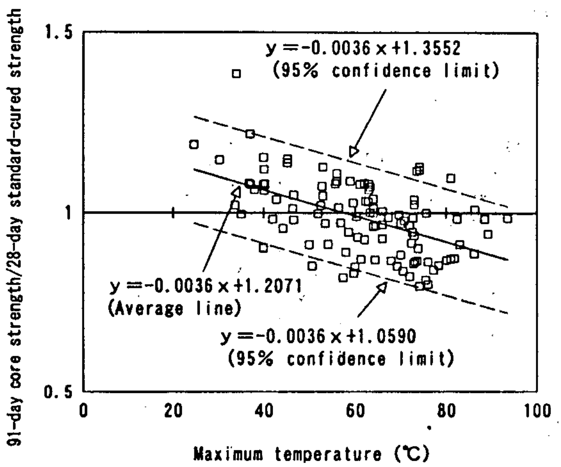

Fig.11 Relationship between 91-day core strength/28-day standard-cured strength and maximum temperature

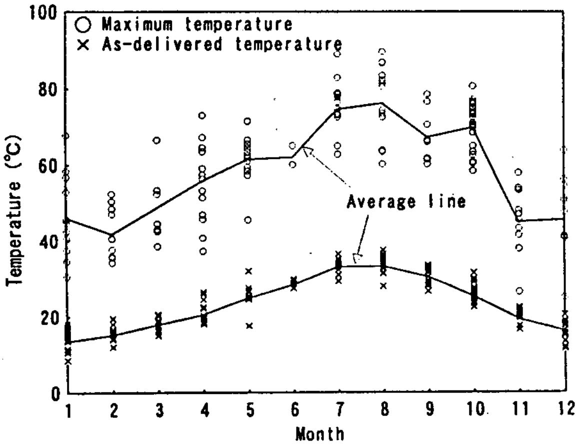

Fig.12 As-delivered temperature and maximum temperature

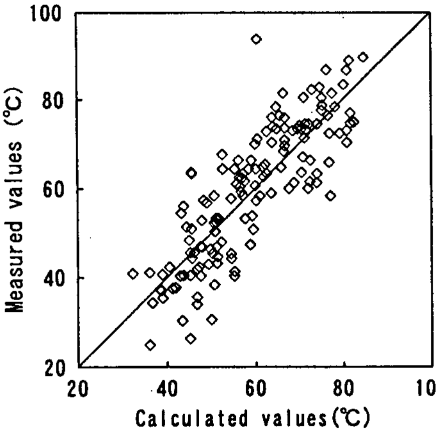

Fig.13 Relationship between calculated and measured maximum temperature

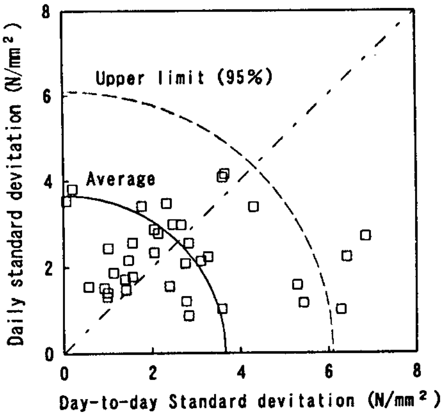

Fig.14 Relationship between daily and day-to-day standard devitations

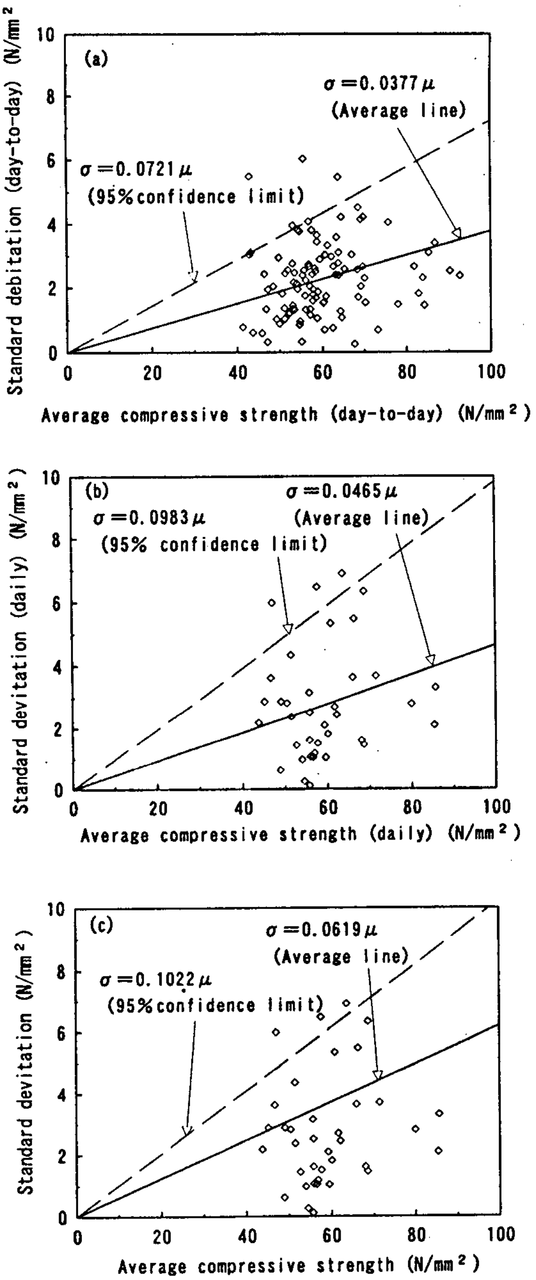

Fig.15 Relationship between average and standard devitation of compressive strengths