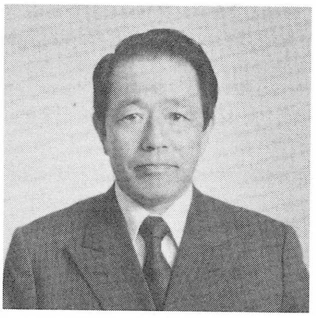
CONCRETE LIBRARY OF JSCE VOL.1, JULY 1983

MECHANICAL PROPERTIES OF REINFORCED CONCRETE MEMBERS AT VERY LOW TEMPERATURES

(Reprint from Proceedings of JSCE, No.285, 1979)



Yukimasa GOTO

Takashi MIURA

SYNOPSIS

Experimental researches on the mechanical properties of reinforced concrete at very low temperatures down to -150°C are reviewed.
The itemes are : (a) Compressive and tensile strengths of concrete, (b) bond characteristics of concrete and deformed reinforcing steel, (c) lapped splice strength of reinforcing bars, (b) flexural strengths and deformation properties of reinforced concrete beams.

Y. Goto is professor of the Tohoku Gakuin University, Sendai, Japan. He was professor of the Tohoku University, Sendai, Japan, till March 1981.
He received his Doctor of Engineering Degree in 1952 from the University of Tokyo. His early research was on the economical design of rigid frame bridges. In recent years, he emphasized studies on the nature of bond and general cracking phenomena in reinforced concrete, durability of concrete structures, and creep of concrete. He was awarded a JSCE prize (Yoshida prize) in 1971 for the study of bond and cracking phenomena in reinforced concrete. He is a member of ACI, JSCE and JCI.

T. Miura is professor of the Tohoku University, Sendai, Japan. He received his Doctor of Engineering Degree in 1980 by this study from the Tohoku University. Also for this study, he was awarded a JSCE prize (Yoshida prize) in 1979. His research interests include properties of reinforced concrete at very low temperatures, durability of cryogenic concrete, durability of reinforced concrete members, and cracking phenomena in steel framed reinforced concrete. He is a member of ACI, JSCE and JCI.

1. Foreword

In recent years in Japan, natural gas has come to be used as city gas and as fuel for thermal power stations, and the quantity of the gas used has been increasing yearly. This is because natural gas has approximately twice the calories of coal gas per unit volume, and when used as city gas, twice the energy can be supplied with existing piping. Also, since the natural gas chiefly used in Japan is not sulfurous, it is advantageous for pollution control in and around large cities.

Because natural gas is gaseous at a normal temperature, its large-volume transportation was limited to pipelines in the past, so that very little had been imported to insular Japan. However, since technology was developed to economically liquefy natural gas, liquefied natural gas, or LNG, has come to be hauled by ship in the same manner as crude oil and stored in tanks in a condition close to normal atmospheric pressure, and large quantities are now being imported.

Meanwhile, the boiling point of LNG is as extremely low as -162°C, and tanks for storage or parts of tankers for hauling it will be exposed to such a low temperature. In general, since thermal insulation materials are lined on the inside surfaces of tanks to come in contact with LNG, the structural materials of the tanks themselves will not be brought down to the same temperature as the boiling point of LNG, but it may be considered that they will become very cold.

In the past, structural materials for structures such as LNG tanks used at low temperatures were special steels made for low temperatures, but reinforced concrete and prestressed concrete have recently increased in number. The reasons for this are that concrete withstands low temperatures well and is more economical than steel, and there have already been many LNG storage tanks made of reinforced concrete. Furthermore, LNG transport tankers are about to be built in the near future with reinforced concrete or prestressed concrete.

Concrete when exposed to low temperature generally changes its nature greatly and cannot be considered to be at the same temperature as at a normal temperature. Consequently, in order to use concrete as a material for a structure which is exposed to such a low temperature, it is important to know the properties of concrete at low temperatures.

The study described in this paper was conducted for the purpose of obtaining data for safe and economical design and construction of concrete structures used under very low temperatures which have recently come to be built in large number and will increase more and more in the future.

The contents of the study are compressive and tensile strengths of concrete, bond characteristics of concrete and deformed reinforcing steel, lapped splice strengths of reinforcing bars, and flexural strengths and deformation properties of reinforced concrete beams at very low temperatures down to about -150°C.

2. Concrete Strength

(1) General

The strength of concrete is a fundamental matter for a concrete structure and must be first investigated thoroughly. Generally speaking, the strength of concrete at low temperature is increased as compared with the strength at normal temperature, and the ratio of increase is greatly influenced by factors such as moisture content in addition to the mix proportions of the concrete.

In this chapter, the manners in which comperssive and tensile strengths of concrete change under low temperatures are investigated with the aim of finding a method of estimating concrete strength at low temperatures from the strength at a normal temperature.

(2) Outline of Experiments

a) Materials Used and Specimens

The cements used for experiments were ordinary Portland cement and high-early-strength Portland cement The fine aggregates were river sands (specific gravity 2.55, absorption 2.32% and 2.38%)and the coarse aggregates crushed stones (specific gravity 2.86, absorption 0.76% and 1.45%). A non-ionic polyoxyethylene alkylaryl ether-type surface active agent was used as an admixture at a rate of 0.06% by weight of cement.

The mix proportions of concrete used are shown in Table 1.

Table 1. Mix Proportions of Concretes Used for Strength Tests

No.	Cement Type	Concrete Type	Max. Size Coarse Agg. (mm)	Slump Range (cm)	Air Content Range (%)	W/C	s/a	Unit Content (kg/m^3)						
								W	C	S	Coarse Agg., G 25mm~15mm	Coarse Agg., G 15mm~10mm	Coarse Agg., G 10mm~5mm	Admix. Chupol C500A(cc)
1	Ordinary	non-AE	25	11±1	1.5±0.5	0.45	41	184	409	703	644	169	316	-
2	Ordinary	non-AE	25	11±1	1.5±0.5	0.50	42	184	368	734	648	170	318	-
3	Ordinary	non-AE	25	11±1	1.5±0.5	0.55	43	184	335	763	646	170	318	-
4	Ordinary	AE	25	11±1	4±0.5	0.45	39	179	397	653	653	172	321	238
5	Ordinary	AE	25	11±1	4±0.5	0.50	40	179	357	682	654	172	321	214
6	Ordinary	AE	25	11±1	4±0.5	0.55	41	179	325	710	653	172	321	195
7	High-Early-Strength	AE	25	11±1	4±0.5	0.50	40	190	380	663	636	167	313	228
8	High-Early-Strength	AE	20	11±1	4±0.5	0.50	38	194	388	622	(20mm~15mm) 342	569	227	233

Regarding shapes and dimensions of specimens, cylinders of 10 cm in diameter and 20 cm length and of 15 cm diameter and 30 cm length were used for measuring compressive strengths and tensile strengths, respectively.

Curing was done by the 11 different methods indicated in Table 2 in order to vary moisture contents of the specimens.

b) Experimentation Method

The method of cooling specimens is shown in Fig. 1. In effect, specimens were placed in a low-temperature chamber made of thermal insulation material, and the temperature in the chamber was lowered by spraying a mist of liquefied nitrogen.

The quantity of liquefied nitrogen sprayed was automatically controlled by an electromagnetic valve to keep the temperature inside the chamber at the specified level. At the same time, a specimen for temperature measurement was also placed inside the chamber and the temperature was lowered gradually so that the temperature difference between the center and surface of the specimen might not become too great.

Loading of specimens was done with a 200 t compression testing machine. Since the temperatures of the specimens would rise from the time when they were taken out of the low-temperature chamber until completion of testing, an allowance was made for the amount of temperature rise of each specimen. Testing times about 2 to 3 minutes with corrections roughly within 5% of the respective temperatures.

Table 2. Varieties of Specimen Curing Methods

No.	Age	1	2	3 ------- 7 ------- 14 15----17 --------- 28 --------- 42	Curing	Cement
1	Concrete Placement	Capping	Demolding, Curing Start	(T)	In Water	Ordinary
2				In Water → (T)	In Water	Ordinary, High-Early
3				In Water → (T)	In Water	Ordinary
4				(T)	Air Drying (Room Temp. 20°C Humidity 50 %)	Ordinary
5				In Air → (T)	Air Drying (Room Temp. 20°C Humidity 50 %)	Ordinary
6				In Air → (T)	Air Drying (Room Temp. 20°C Humidity 50 %)	Ordinary
7				In Air → Oven Drying 110°C → (T)	Oven Drying (28 Days Air Drying Followed by 14 Days Oven Drying)	Ordinary
8				In Air → (T)	Air Drying	High-Early
9				In Air → | → (T)	Oven Drying (28 Days Air Drying Followed by 15 Hours Oven Drying)	High-Early
10				In Air → | Oven Drying → (T)	Oven Drying (28 Days Air Drying Followed by 3 Days Oven Drying)	High-Early
11				In Air → | Oven Drying → (T)	Oven Drying (28 Days Air Drying Followed by 14 Days Oven Drying)	High-Early

(T) denotes strength test performed.

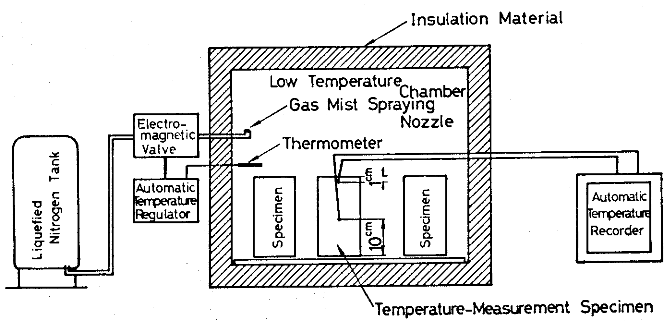


Fig. 1. Specimen Cooling Apparatus

The moisture concrete of concrete was expressed by the ratio of dehydration quantity at the time when dehydration had roughly been completed on placing a subdivided specimen in an oven of + 110°C, to the weight of the dehydrated concrete.

$$\text{Moisture Content (\%)} = \frac{\text{Quantity Dehydrated by Oven of + 110°C}}{\text{Weight of Concrete Dehydrated in Oven}} \times 100$$

It took about 4 days to dehydrate the concrete.

(3) Results of Experiments and Discussions

According to the results of compressive strength measurements the compressive strength of concrete is generally increased as the temperature is lowered.

And the manner in which the increase occurs, vaires with temperatures, mix pro-

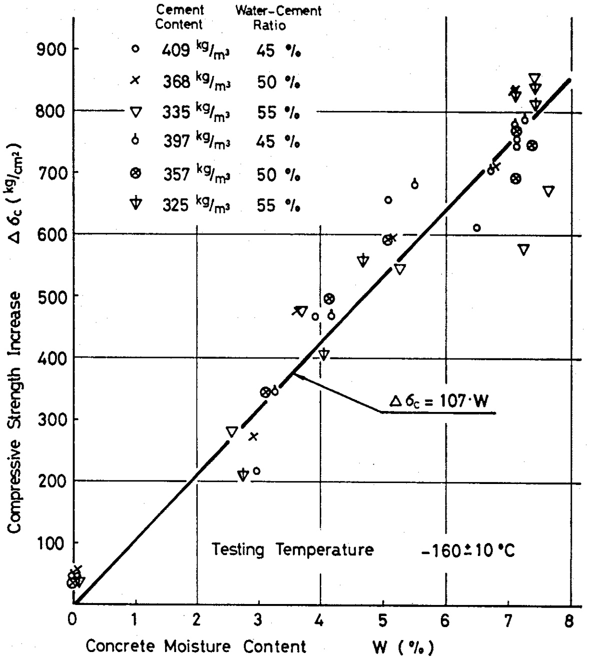


Fig. 2. Compressive Strength Increase of Concrete at -160°C

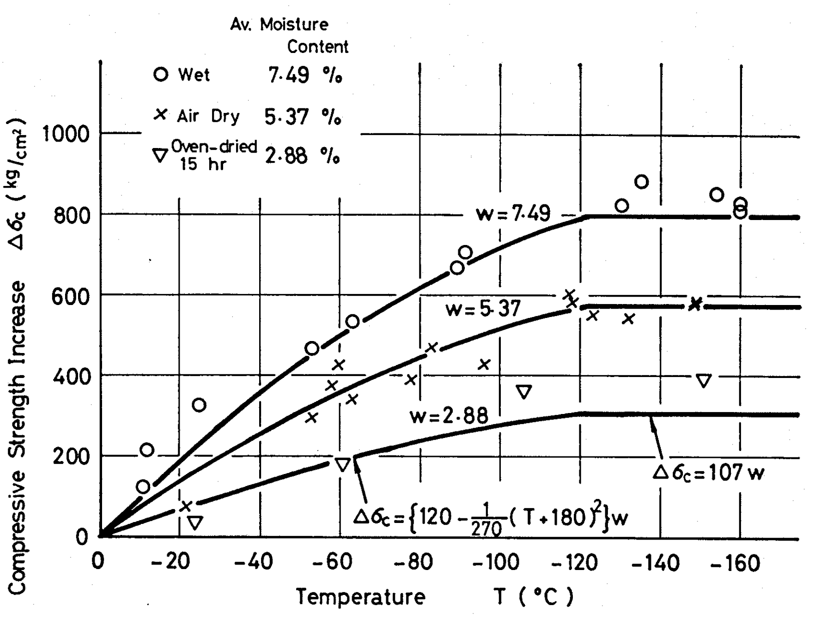


Fig. 3. Relations of Compressive Strength Increase with Temperature and Moisture Concrete

portions and moisture content. The increase in strength from the strength at normal temperature (the difference subtracting the strength at normal temperature from the temperature at low temperature, hereafter it is called "strength increases") does not depend on the mix proportions, but is roughly proportional to the moisture content of that concrete.[1]

The relations between compressive strength increases and moisture contents at -160°C of concretes of various mix proportions are indicated in Fig. 2. These relations determined by the method of least squares results in $\Delta\sigma_c = 107w$, where $\Delta\sigma_c$ is compressive strength increase (kg/cm^2), and w is moisture content (%) of concrete.[2]

These relations expressed for various temperatures are as shown in Fig. 3, and the relation between compressive strength increase of concrete and moisture content of concrete may be approximated by the equation below [3].

Provided that $\Delta\sigma_c \leqq 107 \cdot w$,

$$\Delta\sigma_c = \left\{120 - \frac{1}{270}(T+180)^2\right\} \cdot w \qquad (1)$$

where T is concrete temperature (°C).

As a result of the above, if the strength and moisture content of the concrete at normal temperature is known, the strength at low temperature may be estimated by the following equation:

$$\sigma_{cL} = \Delta\sigma_c + \sigma_{cO} \qquad (2)$$

where σ_{cL} is concrete strength at low temperature and σ_{cO} is concrete strength at normal temperature.

As can be seen in Fig. 3, the strength of concrete generally increases with lowering of temperature and it is considered the reason for this is freezing of free water in the voids of concrete and it changes the voids which was occupied by liquid water into solids which have strength. Further, with temperature in the vicinity of -120°C the condition of increase changes and scatter increases, and it is thought to be because the state of the ice in the concrete changes in this vicinity. [4]

The tensile strength of concrete, as well as compressive strength at low temperature is shown in Fig. 4. in which more or less the following relation is thought to exist. [5]

$$\sigma_T = 0.38\sigma_c^{3/4} \qquad (3)$$

where, σ_T is tensile strength of concrete (kg/cm^2), and σ_c is compressive strength of concrete (kg/cm^2).

The above equation is considered to be applicable to all concretes of water-cement rations of around 0.45 to 0.55 generally used, and although the number of experiments is small, it is known to roughly fit even very high strength concrete of compressive strength at normal temperature of about 800 kg/cm^2.

3. Characteristics of Bond to Deformed Bars and Strengths of Lapped Splices

(1) General

Reinforced concrete is a combination of concrete strong against compression and

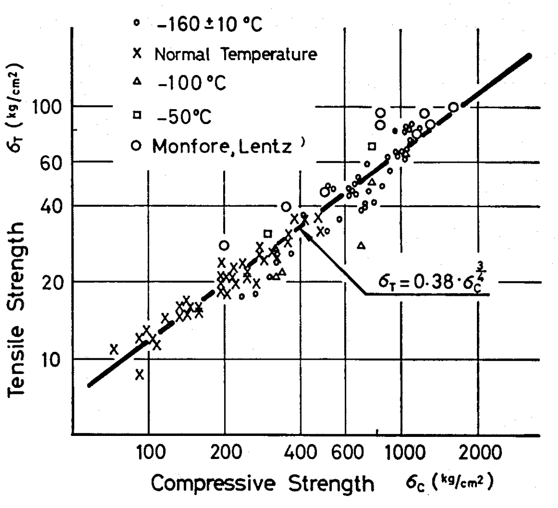


Fig. 4. Relation Between Tensile and Compressive Strengths

weak against tension, and reinforcing steel strong against tension, and is combined into a material, working in a body. Consequently, it is generally considered that reinfrocement and concrete are in a thoroughly bonded state, and within the range of design load, there is no slippage as a whole except for local spots near cracks.

Crack dispersion and anchorage are generally considered as properties related to bond between concrete and reinforcement. Crack dispersion indicates the condition (whether cracks of large width are produced in small number, or cracks of small width are produced in large number) of cracks which occurs in concrete in the direction perpendicular to the reinforcing bar axis when tensile stresses are produced in a reinforced concrete member, and generally, it is thought better for cracks of small width to be produced in large number. Anchorage properties express the force transmission capacities of the reinforcement and concrete at the end portions of bars.

Lapped splices are Joints which transmit forces acting on one reinforcing bar to the other through the medium of the surrounding concrete, and it may be considered that the influence of bond between reinforcement and concrete is great.

In this way, the bond between reinforcement and concrete is very important in order to materialize the combination of reinforcement and concrete. On the other hand, it may be considered that the properties of concrete and reinforcement will change greatly at low temperature compared with those at normal temperature, and that the condition of bond will also vary.

So the experimental investigations were made concerning the bond between deformed bars and concrete and are described in this chapter.

(2) Outline of Experiments

a) Materials Used

The concrete used for the experiments was of identical type as No.7 in Table 1.

Table 3. Properties of Reinforcing Bars Used for Bond Tests

Reinfrocing Bar	SD 30		SD 30L	
Testing Temperature	Normal Temp.	−160°C	Normal Temp.	−160°C
Tensile Strength (kg/mm^2)	58	71	47	66
Yield Point (kg/mm^2)	38	—	37	59
Elongation (Reference Point Spacing 20 cm) (%)	25	—	22	24

The concrete was cast in a constant temperature room (temperature 20°C, humidity 50%), stripped at the age of 2 days, and cured in water of temperature 20±1°C.

The experiments were conducted at 7 day age.

The reinforcing bars used were two kinds of commercial SD30 and a bar equivalent to SD30 but manufactured on a trial basis for low temperature (hereafter indicated as SD30L). Both were D22 deformed bars having diagonal lugs.

The properties of the reinforcing bars used in the experiments are given in Table 3.

b) Specimens and Experimentation Method

1 Crack Dispension Property Tests

Axially tensioned prism specimens having square cross sections were used in order to investigate crack dispersion properties. The specimens, as shown in Fig. 5, were of two kinds having cross section sides of 10 cm and 12 cm. The notch for low temperature differs in structure from that for normal temperature.

This is because galvanized iron sheet and concrete are frozen together at low temperature, and a notch of a structure for normal temperature will not work as a notch. The reinforcing bars used were SD30 for tests at normal temperature and SD30L for tests at low temperature.

The loading apparatus is shown in Fig. 6.

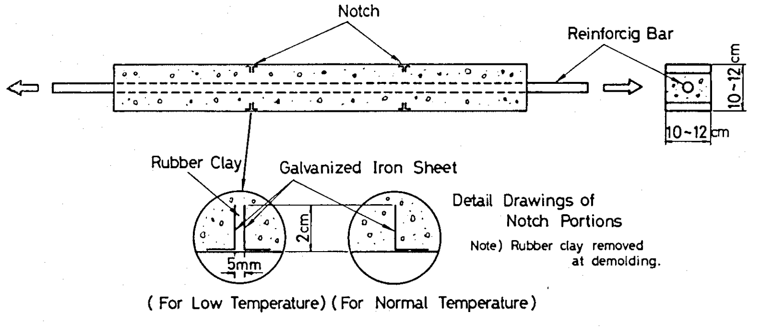


Fig. 5. Specimen for Crack Dispersion Property Test

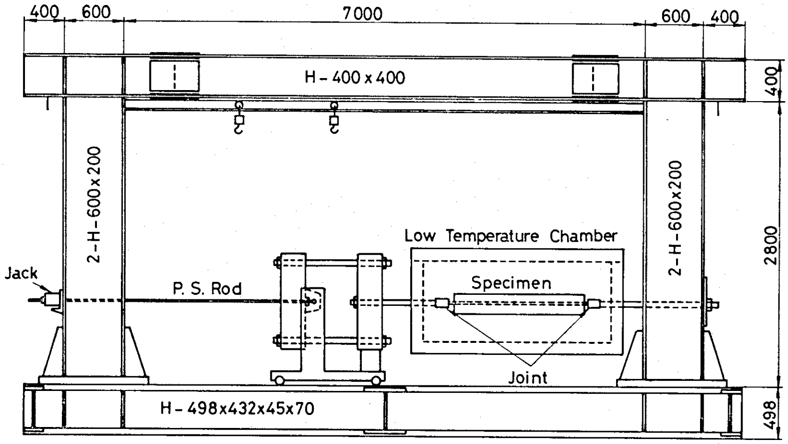


Fig. 6. Crack Dispersion Property Testing Apparatus

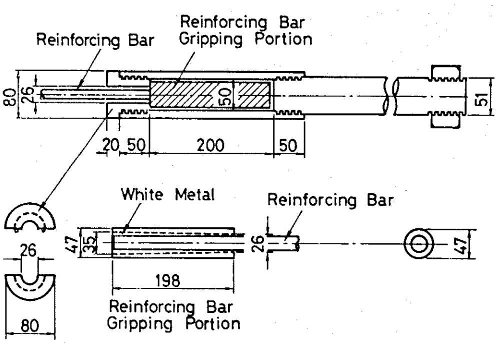


Fig. 7. Joint Structure of Reinforcement in Low-Temperature Chamber

Since the strength of concrete at low temperature would differ according to the moisture content of the concrete, all of the tests were performed in a wet condition (moisture content about 7%) immediately after they were taken out from the water-curing tank.

As the strength of reinforcement at low temperature is large than at normal temperature, it is necessary for the cross section of the part exposed outside the low-temperature chamber to be made large. Therefore joints such as one indicated in Fig. 7 were provided in the low temperature chamber to prevent rupture of bars.

2 Anchorage Tests

As anchorage tests, two kinds of failure mainly through splitting of concrete (A) shown in Fig. 8. and mainly through slip-out of reinforcement from concrete (B) shown in Fig. 9, were carried out.

In general, the factors thought to affect the anchroage of a deformed bar are bar diameter, bar surface configuration, reinforcement ratio, cover, anchorage length, ratio between shear span and effective height, mechanical properties of reinforcing bar, and various strengths of concrete, and in these experiments, specimen size, cover and materials used were maintained constant with only anchorage length varied, and tests were performed at normal temperature and

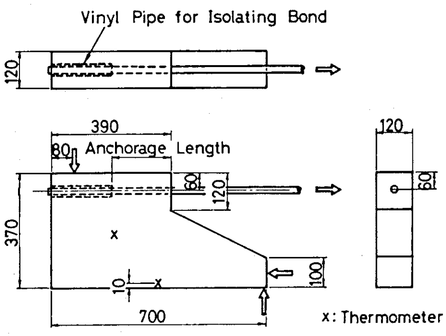


Fig. 8. Specimen for Anchorage Test (A)

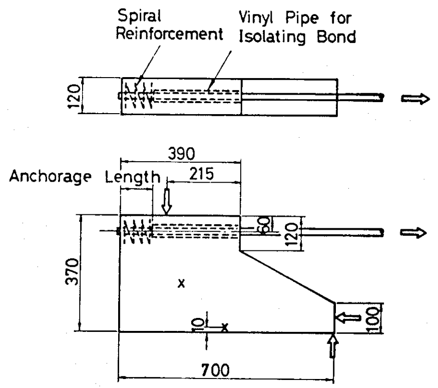


Fig. 9. Specimen for Anchorage Test (B)

-160°C for comparison studies.

The loading apparatus is shown in Fig. 10.

3 Lapped Splice Tests

The specimens used in the experiments on lapped splices were those shown in Fig. 11. with joints symmetrically located at two places; there were two kinds; the specimens had cross sections of width of 38 cm and thickness of 10 cm (cover 3.9 cm), and those of width of 40 cm and thickness of 12 cm (cover 4.9 cm). The experiments were carried out by varying lapped lengths.

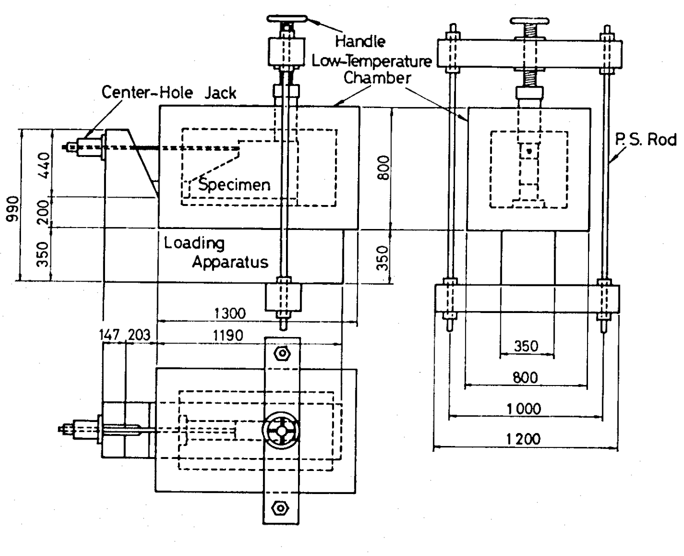


Fig. 10. Loading Apparatus for Anchorage Test

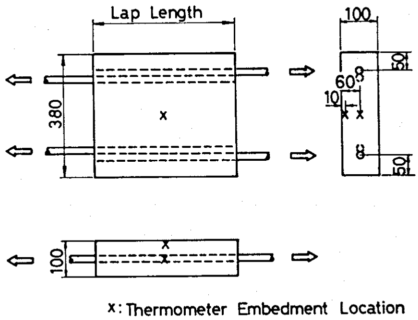


Fig. 11. Specimen for Lapped Splice Test

The loading apparatus is shown in Fig. 12.

4 Method of Cooling Specimens
Cooling of specimens was done by spraying liquid nitrogen into the low-temperature chamber. Control of temperature was accomplished using automatic and manual devices in a manner that the temperature difference between the centers and surfaces of specimens may not be more than 20°C.

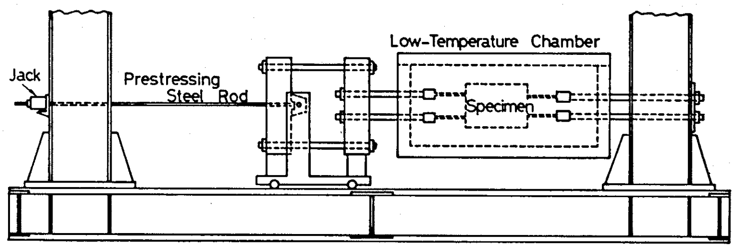


Fig. 12. Loading Apparatus for Lapped Splice Test

Loading was done in a condition of specimens kept uniformly at the specified temperature. In the crack dispersion tests, the top lid of the low-temperature chamber was opened to confirm cracked conditions, and the temperature rise by it was about 10°C.

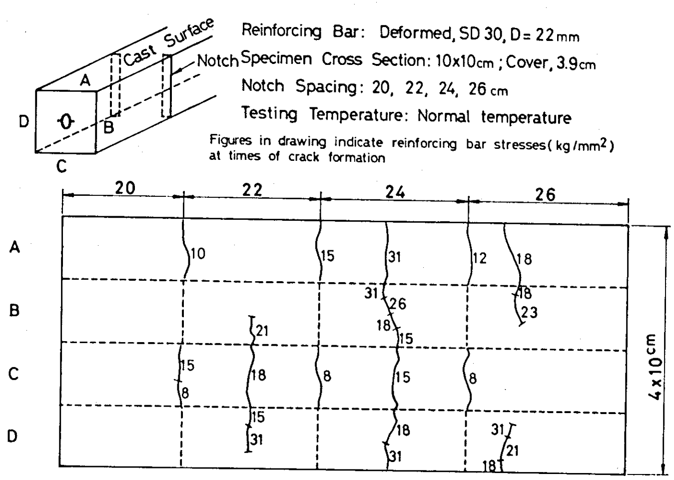


Fig. 13. Example of Results of Crack Dispersion Property Tests (Normal Temperature)

(3) Results of Experiments and Discussion

a) Crack Dispersion Property Tests
Examples of the results of crack dispersion property tests are shown in Fig.13. and Fig. 14. These results are given is summarized form in Table 4.

According to these results, there was not very much scatter in case of normal temperature for results of experiments with specimens having cover of 3.9 cm, and it is seen that the maximum spacing of lateral cracks produced in the direction perpendicular to reinforcing bar axis is from 20 cm to 22 cm. As contrasted with this, there is considerable scatter in case of -160°C, and it is thought the maximum crack spacing will be about 30 cm to 36 cm.

Further, according to the results of experiments for cover of 4.9 cm,whereas the maximum crack spacing at normal temperature is from 25 cm to 30 cm, that at -160°C was about 55 cm to 59 cm.

In a summary it may be seen that the maximum crack spacing of reinforced concrete is 40% to 80% larger at -160°C compared with normal temperature, and in some cases, even larger.

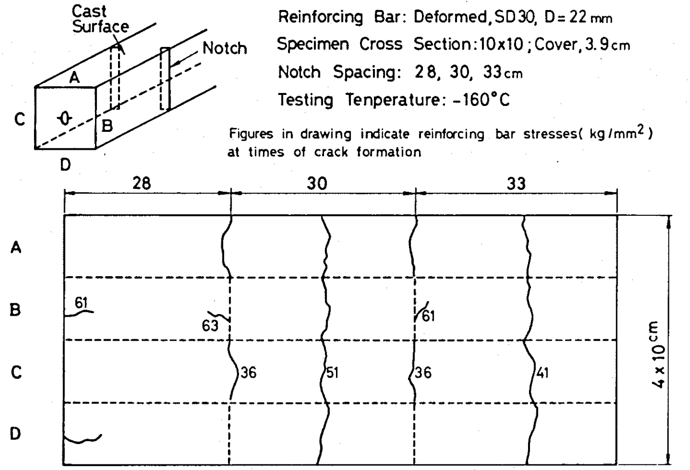


Fig. 14. Example of Results of Crack Dispersion Property Tests (-160°C)

Table 4. Crack Dispersion Property Test Results

Normal Temperature

Re bar \ Notch Spacing (cm)	18	20	22	24	26
SD 30	○ ○	○ ○ ○ ○ ○ ○	○ ○ ● ◑ ● ●	● ● ● ● ● ●	● ● ● ● ● ◑

-160°C

Re bar \ Notch Spacing (cm)	25	28	30	33	36	38	40	45	50	54	56	58	60	80
SD 30 L		○	○ ○ ○ ○ ○	○ ○ ◑ ○	○ ● ● ○ ○ ●	◑ ●	○ ● ●	◑	●	● ●	●	○	○	
SD 30	○ ○	○ ○ ○ ○ ○ ○	○ ○ ◑ ○ ○ ○ ○ ○	◑ ○ ● ◑ ○ ○ ◑ ●	◑ ◑		●		●	●			●	●

Specimen cross section, 10cm x 10cm
Cover, 3·9 cm

○ : Uncracked specimen
● : Specimen with cracking between notches
◑ : Specimen of borderline case

The reason why maximum crack spacing is increased under low temperature may be considered to be as described below.

The maximum crack spacing L of lateral cracks is roughly of the relation indicated in the following equation.

$$\sigma_T = \frac{\pi D}{A_c} \int_0^{L/2} U_x \, dx$$

where,

σ_T : tensile strength of concrete
D : reinforcing bar diameter
A_c : concrete cross section
U_x : bond stress at location x from crack$=U_m f(x/L)=U_m f(\phi)$
U_m : maximum unit bond stress

Therefore:

$$L = \frac{A_c \sigma_T}{DU_m \int_0^{1/2} f(\phi)\, d\phi} = \frac{2A_c \sigma_T}{\pi D\bar{U}} \qquad (4)$$

where,

$\bar{U}$: average value of bond stress intensity at spacing of $L/2$.

It is learned from the above equation that the maximum crack spacing is proportional to $\sigma_T/\bar{U}$. Meanwhile, the bond rigidity of concrete is increased when the temperature is lowered, and it is estimated that the distribution from of bond stress in the reinforcing bar axial direction will approach the from of a triangular distribution with lateral cracking as the base.

Therefore, when L is long, the increase in $\bar{U}$ becomes relatively small compared with the increase in σ_T, and it is considered that the value of $\sigma_T/\bar{U}$ will become larger than at normal temperature. Further, as L becomes larger, in effact, as the concrete cross section becomes larger (cover becomes larger), such influence will become more prominent, and therefore, when cover is thick, the difference in maximum crack spacing between low and normal temperatures will become greater.

b) Anchorage Tests

The results of anchorage tests (A) are shown in Table 5 and Fig. 15. [6) 7)]. From these, it may be seen that bond strengths at the various anchorage lengths at -160°C will be roughly 2.2 to 2.6 times the case of normal temperature.

On the other hand, since it is known that anchorage strength is roughly proportional to tensile strength of concrete, the ratio of tensile strength obtained by calculations will be as indicated below.

1 The average compressive strength and moisture content of concrete at normal temperature are σ_c=328 kg/cm^2 and 7.52%, respectively.
2 From Eqs. (1) and (2), the compressive strength at -160°C will be 328 (kg/cm^2)+107×7.52=1133 (kg/cm^2)
3 The tensile strengths of concrete at normal temperature and -160°C , according to Eq. (3), are 29 kg/cm^2 at normal temperature and 74 kg/cm^2 at -160°C, the ratio between them being 74/29≒2.5
In other words, it was found that the experimental values more or less coincided with the calculated values.

The results of anchorage tests (B) are given in Table 6. According to this table, the strengths are much greater compared with the anchorage tests (A), but

Table 5. Example of Results of Anchorage Tests (A), (Cover 4.9 cm)

Anchorage Length (cm)	No.	Normal Temperature (SD 30) Failure Load (t)	Re. bar Stress at Failure (kg/mm²)	Av. Bond Stress (kg/cm²)	−160°C (SD 30L) Failure Load (t)	Re. bar Stress at Failure (kg/cm²)	Av. Bond Stress (kg/cm²)	Low Temp. Value / Norm. Temp. Value
25.0	1	9.4	24.3	53.7	20.0	51.7	114	
	2	9.6	24.8	54.9	20.0	51.7	114	
	3	9.4	24.3	53.7	22.0	56.8	126	
	Av.	9.5	24.4	54.0	20.7	53.4	118	2.19
12.5	1	7.1	18.3	81.1	15.4	39.8	176	
	2	5.6	14.5	64.0	16.9	43.7	193	
	3	6.0	15.5	68.6	16.5	42.6	189	
	Av.	6.2	16.1	71.2	16.3	42.0	186	2.61
6.25	1	3.8	9.8	86.9	8.4	21.7	192	
	2	3.8	9.8	86.9	9.4	24.3	215	
	3	3.8	9.8	86.9	8.7	22.5	199	
	Av.	3.8	9.8	86.9	8.9	22.9	203	2.34

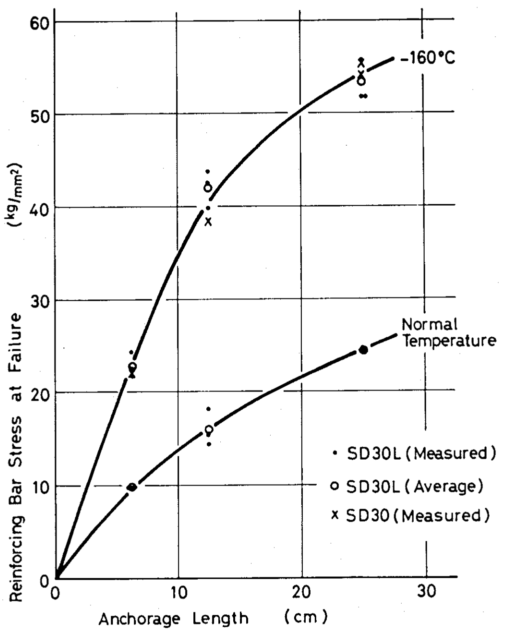


Fig. 15. Results of Anchorage Tests (A)

the ratio between low and normal temperature values is not changed very much, and it is thought a trend similar to that for tensile strength of concrete is indicated in this case, too.

c) Lapped Splice Tests

The relation between lapped splice strength and testing temperature is indicated in Fig. 16.[8] The relation between lapped splice strength and lap length are given in Fig. 17. The relations between lapped splice strength ratio and lapped splice length are indicated in Fig. 18.

Table 6. Results of Anchorage Tests (B)

Anchorage Length (cm)	No.	Normal Temperature (SD 30) Failure Load (t)	Re. bar Stress at Failure (kg/mm^2)	Av. Bond Stress (kg/cm^2)
10	1	14.5 (Bar Yielded)	37.4	-
5	1	9.2	23.7	263
	2	9.9	25.5	282
	3	8.9	23.1	254
	Av.	9.3	24.1	266

Anchorage Length (cm)	Low Temperature (SD 30) Testing Temp. (°C)	Failure Load (t)	Re. bar Stress at Failure (kg/mm^2)	Av. Bond Stress (kg/cm^2)	Low Temp. Value / Norm. Temp. Value*
10	-160	27.0 (Bar Yielded)	69.8	-	-
5	-160	22.5	58.1	643	2.42
	-168	25.5	64.6	714	2.68

* Average values for normal temperature values.

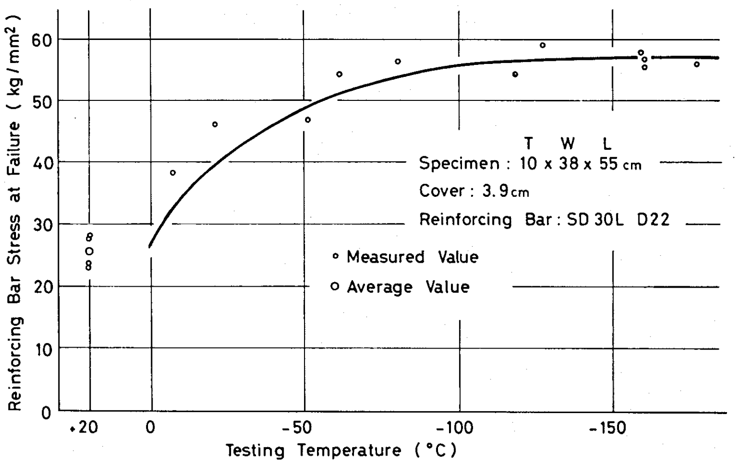


Fig. 16. Relation Between Lapped Splice Strength and Testing Temperature

The lapped splice strength for each lap length may be obtained from Fig. 17. The part of the broken line in the graph indicates where failure could not be induced because of limitations of the testing apparatus and was estimated using the relation of Fig. 18.

According to Fig. 18, the ratio between strengths at -160°C and normal temperature is not related very much to thickness of cover, but is lowered as lap length becomes greater. In effect, it may be imagined that the degree of bond stress concentration due to increased lap length becomes greater at low temperature than at normal temperature. The reasons for this are considered to be the increase in rigidity of concrete at low temperature similarly to what was described under crack dispersion tests, or the influence of stress concentration which became prominent with elongation of concrete after yielding had become smaller at low temperature.

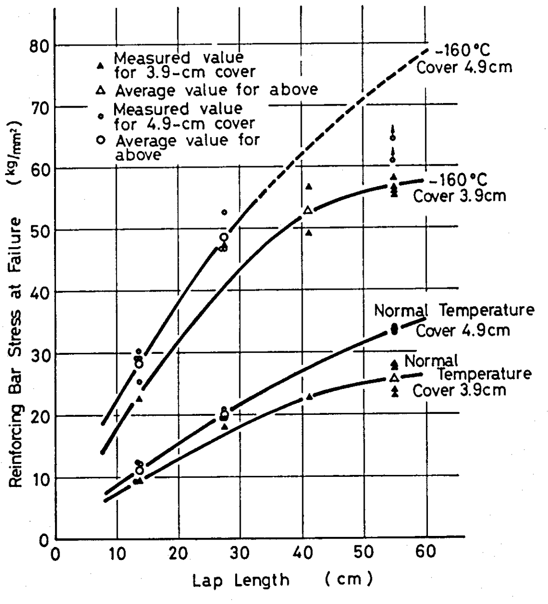


Fig. 17. Relation Between Lapped Splice Strength and Lap Length

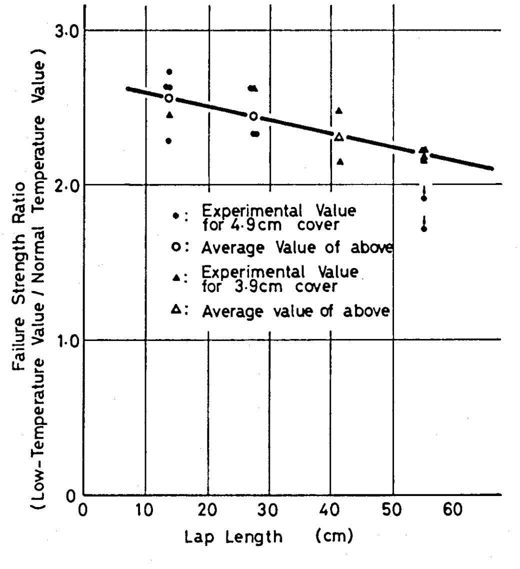


Fig. 18. Relation of Lapped Splice Strength Ratio for -160°C and Normal Temperature to Lap Length

In any event, it may be surmised from these considerations that at low temperature the anchorage strength of a deformed bar to concrete or lapped splice strength, increased at a ratio roughly equal to the ratio in tensile strength increase of the concrete, whereas in contrast, when the length becomes greater the ratio of increase is lowered. Accordingly, it is thought that in designing a reinforced concrete structure to be used at low temperature, it will be necessary to give thorough consideration to these points.

4. Bending Properties of Reinforced Concrete Beams Under Very Low Temperature

(1) General

When reinforced concrete is used as material for LNG storage tanks or LNG transport tankers, tanks in an earthquake or tankers in collisions or other accidents must have ample ductility along with strength. It is said that ductility is also necessary to warning against failure of a structure or for redistribution of stresses.

It is thought the strength (principally flexural strength) and ductility of a reinforced concrete member at low temperature can be estimated to an extent if the properties of concrete and reinforcing steel are thoroughly understood, but these properties under very low temperatures have not yet been adequately clarified. Therefore, the ultimate bending moment and ductility at the bending of reinforced concrete members under low temperature obtained through experiments are described, comparisons are made with values at normal temperature, while also, the method of estimating strength of reinforced concrete used at low temperature and the method of imparting ample ductility to the reinforced concrete are studied in this chapter.

(2) Outline of Experiments

a) Materials Used
The cement used in the experiments was high-early-strength portland cement, fine aggregate was river sand (specific gravity 2.52, absorption 2.50%), coarse aggregate was crushed stone (specific gravity 2.86, absorption 0.76%), while a polyoxyethylene alkyl allyl ether type non-ionic surface active agent was used as an admixture 0.06% by weight of cement.

The concrete used for the experiments was of identical type as No.8 in Table 1.

After casting of concrete, molds were stripped at the age of 1 day, and specimens for air-dry curing were placed in a constant-temperature room of air temperature of 20°C and humidity of 50%, while those for underwater curing were cured in a constant-temperature tank of water temperature of 20°C. The testing ages were 14 to 17 days owing to circumstances of testing.

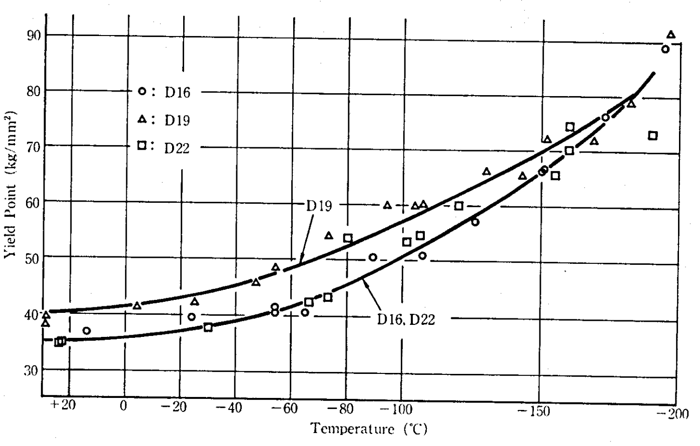


Fig. 19. Yield Point of Reinforcing Bar Used vs. Temperature

The reinforcing bars used were the three SD30 varieties of D16, D19 and D22, the results of measurements of the respective yield points being indicated in Fig. 19. These values were all arranged based on the cross-sectional areas of the bars measured (D16: 1.90 cm^2; D19: 2.79 cm^2; D22: 3.80 cm^2). The results of measurement of Young's moduli at various temperatures are shown in Fig. 20.

b) Specimens and Method of Experimentation

The specimens used for experiments had single reinforcement of span of 1.5 m and length of 1.7 m with various reinforcement ratios. Further, since the properti-

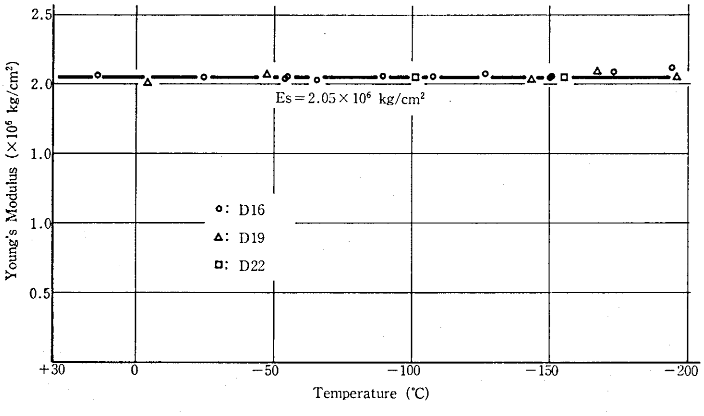


Fig. 20. Young's Modulus of Reinforcing Bar Used vs. Temperature

es would differ with the moisture content of concrete, experiments were conducted on two kinds of specimens, moist and air-dry. The dimensions and reinforcement quantities of the specimens are shown in Figs. 21. and 22. Loading was third point loading at two points with load applied from outside while the specimen to be tested was inside a low-temperature chamber (see Fig. 23.). The items of measurement were load, deflection, concrete strain and specimen temperature. Deflection of the beam was measured by relative movements of rods sticking out side the low-temperature chamber from the support points and the middle of the beam (see Fig. 24.). Confirmation was made that deflection values determined by this method accurately express actual deflections of beams.

For the method of loading, pulsating low-cycle repetitive loading was considered, but as a result of comparisons of repetitive loading and non-repetitive static monotonous incremental loading respectively with Specimen No. 1 (p=2.42%) and Specimen No. 2 (p=4.94%) at the 4 temperature levels of normal temperature, -50°C, -100°C and -150°C, a total of 8 conditions, since it was found that roughly the same deflection behaviors were indicated, in subsequent experiments only static loading was done. Examples of the results are shown in Fig. 25. and Fig. 26.

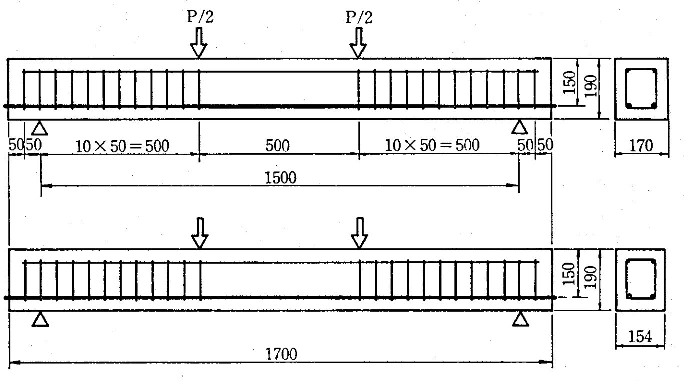


Fig. 21. Dimensions of Beam Specimens

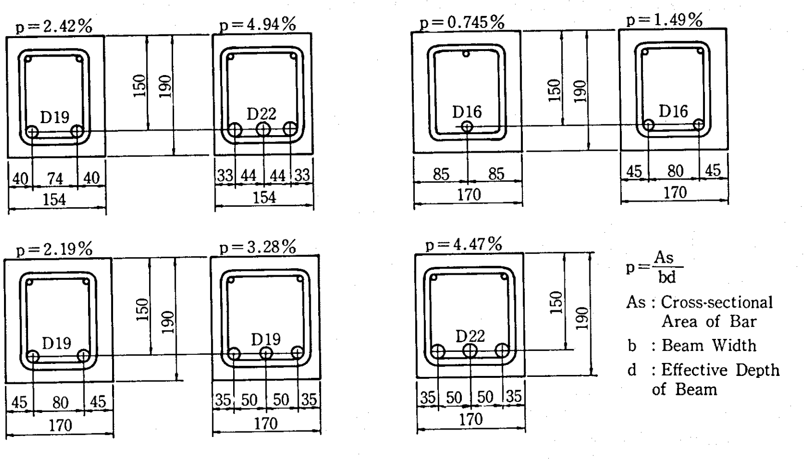


Fig. 22. Cross Sections and Reinforcement Quantities of Beam Specimens

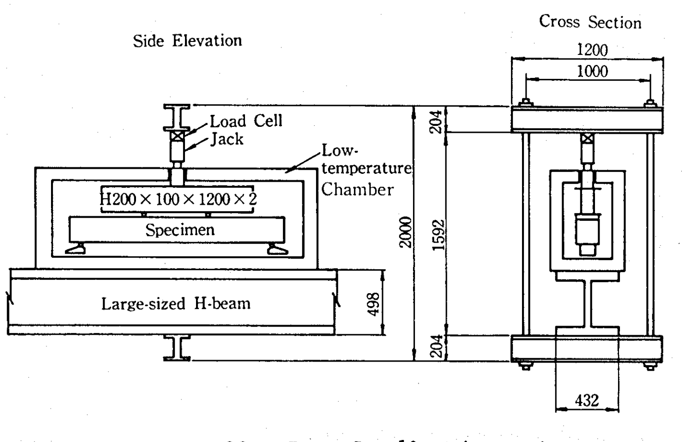


Fig. 23. Beam Loading Apparatus

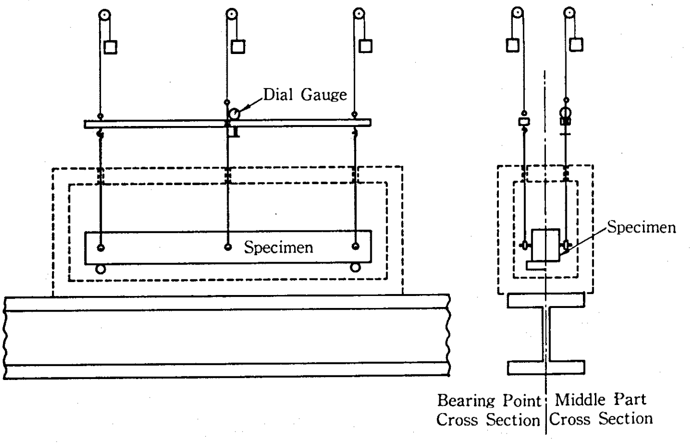


Fig. 24. Beam Deflection Measurement Apparatus

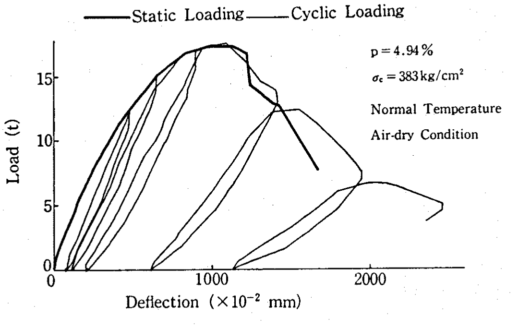


Fig. 25. Example of Comparison of Cyclic Loading and Static Loading (Normal Temperature, p=4.94%)

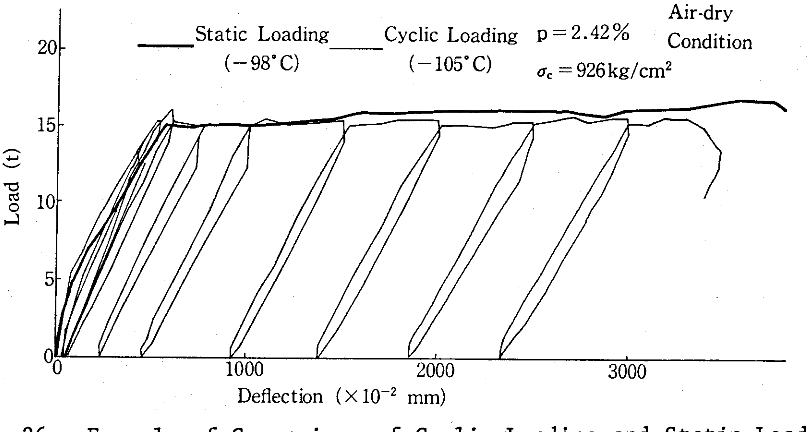


Fig. 26. Example of Comparison of Cyclic Loading and Static Loading (-100°C, p=2.24%)

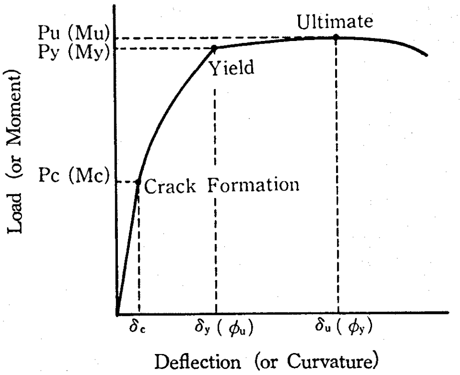


Fig. 27. Typical Diagram of Load-Deflection Curve

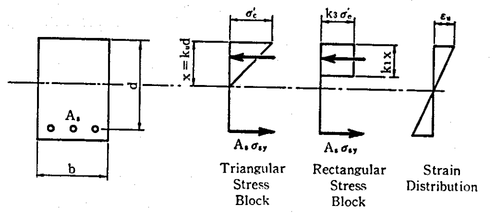


Fig. 28. Types of Stress Blocks Assumed

Table 7. Results of Strength Tests of Reinforced Concrete Beams

Yielding moments in parentheses — Air-dry

Temp. °C	Reinforcement Ratio p (%)	Measured Ultimate Bending Moment M_u (t-m)	Method Assuming Triangular Stress Block: Calculated Moment M_{uc}	Method Assuming Triangular Stress Block: M_u/M_{uc}	Method Assuming Rectangular Stress Block: Calculated Moment M_{uc}	Method Assuming Rectangular Stress Block: M_u/M_{uc}
Normal Temp.	0.745	(0.900) 1.13	0.969	(0.929) 1.17	0.974	(0.924) 1.16
	1.49	(1.93) 2.17	1.84	(1.05) 1.18	1.86	(1.04) 1.17
	2.19	2.88	2.72	1.06	2.78	1.04
	3.28	3.91	3.71	1.05	3.84	1.02
	4.47	4.56	4.26	1.07	4.46	1.02
- 50	1.49	(2.23) 2.65	2.15	(1.04) 1.23	2.17	(1.03) 1.22
- 52	2.19	3.40	3.55	0.959	3.59	0.946
- 50	3.28	4.96	4.89	1.02	4.99	0.995
- 50	4.47	5.46	5.64	0.968	5.79	0.943
- 99	1.49	(2.68) 3.25	2.71	(0.989) 1.20	2.73	(0.982) 1.19
-105	2.19	4.33	4.41	0.982	4.47	0.970
-104	3.28	5.85	6.21	0.942	6.34	0.922
-103	4.47	6.85	7.24	0.947	7.43	0.922
-149	0.745	(1.63) 2.03	1.82	(0.896) 1.11	1.83	(0.891) 1.11
-150	1.49	(3.80) 3.88	3.53	(1.08) 1.10	3.56	(1.07) 1.09
-148	2.19	4.83	5.23	0.924	5.30	0.911
-149	3.28	7.68	7.40	1.04	7.56	1.02
-149	4.47	8.73	9.04	0.966	9.31	0.938

Table 7. (Continued) Moist

Temp. °C	Reinforcement Ratio p (%)	Measured Ultimate Bending Moment M_u (t-m)	Method Assuming Triangular Stress Block: Calculated Moment M_{uc}	Method Assuming Triangular Stress Block: M_u/M_{uc}	Method Assuming Rectangular Stress Block: Calculated Moment M_{uc}	Method Assuming Rectangular Stress Block: M_u/M_{uc}
Normal Temp.	0.745	(1.05) 1.30	0.972	(1.07) 1.34	0.978	(1.07) 1.33
	1.49	(1.93) 2.31	1.87	(1.03) 1.23	1.89	(1.02) 1.22
	2.19	3.00	2.81	1.07	2.85	1.05
	3.28	4.08	3.91	1.04	4.01	1.02
	4.47	4.85	4.55	1.06	4.71	1.03
- 52	1.49	(2.20) 3.00	2.19	(1.00) 1.37	2.20	(1.00) 1.36
- 53	2.19	(3.38) 4.10	3.65	(0.926) 1.12	3.69	(0.915) 1.11
- 61	3.28	5.73	5.45	1.05	5.53	1.04
- 50	4.47	6.05	5.80	1.04	5.93	1.02
-105	1.49	(2.80) 3.48	2.85	(0.982) 1.22	2.87	(0.976) 1.21
104	2.19	(4.25) 5.00	4.53	(0.938) 1.10	4.57	(0.930) 1.09
-107	3.28	6.93	6.60	1.05	6.70	1.03
-105	4.47	7.95	7.77	1.02	7.91	1.01
-119	1.49	(3.03) 3.81	3.05	(0.993) 1.25	3.07	(0.987) 1.24
-137	1.49	(3.35) 3.90	3.35	(1.00) 1.16	3.37	(0.994) 1.16
-150	0.745	(2.10) 2.47	1.84	(1.14) 1.34	1.85	(1.14) 1.34
-154	1.49	(3.80) 4.05	3.66	(1.04) 1.11	3.68	(1.03) 1.10
-152	2.19	(5.38) 5.63	5.37	(1.00) 1.05	5.43	(0.991) 1.04
-150	3.28	7.75	7.59	1.02	7.73	1.00
-150	4.47	9.28	9.35	0.99	9.58	0.97

(3) Results of Experiments and Discussions

a) Strengths of Reinforced Concrete Beams
A part of the experimental results is indicated in Table 7.

The ultimate bending moment here, as shown in Fig. 27, means the bending moment, immediately before the load-deflection curve after reaching a maximum beings to be lowered. Calculations of ultimate bending moments were made by assuming the two kinds of stress blocks below (see Fig. 28.).

(1) Triangular Stress Block

$$\frac{bx\sigma_c'}{2} = A_s\,\sigma_{sy}\,.$$

$$\therefore\quad x = \frac{2A_s\,\sigma_{sy}}{b\sigma_c'} = \frac{2p\sigma_{sy}\,d}{\sigma_c'}\,. \tag{5}$$

$$M_u = A_s\,\sigma_{sy}\Big(d - \frac{x}{3}\Big) = A_s\,\sigma_{sy}\,d\Big(1 - \frac{2p\sigma_{sy}}{3\sigma_c'}\Big)\,. \tag{6}$$

(2) Rectangular Stress Block (Values of ACI Standards Used)

$$k_1k_3x\sigma_c'b = A_s\,\sigma_{sy}\,.$$

$$\therefore\quad x = \frac{p\sigma_{sy}\,d}{k_1k_3\sigma_c'}\,. \tag{7}$$

$$M_u = A_s\,\sigma_{sy}\Big(d - \frac{k_1x}{2}\Big) = A_s\,\sigma_{sy}\,d\Big(1 - \frac{p\sigma_{sy}}{2k_3\sigma_c'}\Big)\,. \tag{8}$$

where

- b : width of beam (cm)
- σ_c': compressive strength of standard specimen of concrete (kg/cm^2)
- A_s: cross-sectional area of reinforcing bar (cm^2)
- ρ_{sy}: yield strength of reinforcing bar (kg/cm^2), (determined from Fig. 19)
- p : reinfrocement ratio$=A_s/bd$
- d : effective depth of beam (cm)
- k_1 : from ACI standard, $k_1=0.85$
  (when $\sigma_c' \leqq 280$ kg/cm^2)
  $=0.85-0.05\,((\sigma_c'-280)/70)$
  (when 280 kg/cm$^2 < \sigma_c' \leqq 560$ kg/cm^2)
  $=0.65$
  (when $\sigma_c' > 560$ kg/cm^2, assumed to be constant as stress-strain curve approaches straight line)
- k_3: similarly, $k_3=0.85$

On examining the results in Table 7, it may be seen that the calculation results coincide well with the measured values regardless of whether stress blocks are assumed to be triangular or rectangular.[10] However, with beams of particularly low reinforcement ratios, the calculated values are closer to the measured values of yield bending moment rather than ultimate bending moment. The reason for this is thought to be that with specimens of reinforcement ratios too small, the deflections of actual beams at ultimate bending moment will be extremely large, and the reinforcing bars enter the range of strain hardening and stresses become considerably greater than yield stresses hypotehesized in calculations.

b) Ductility of Reinforced Concrete Beams

As an index for expressing the ductility of a reinforced concrete beam, the ratio δ_u/δ_y of ultimate deflection δ_u and yield deflection δ_y when the beam is deformed on being subjected to load is used, and this will be termed "ductility factor due to deflection."

The results are shown in Figs. 29. and 30.

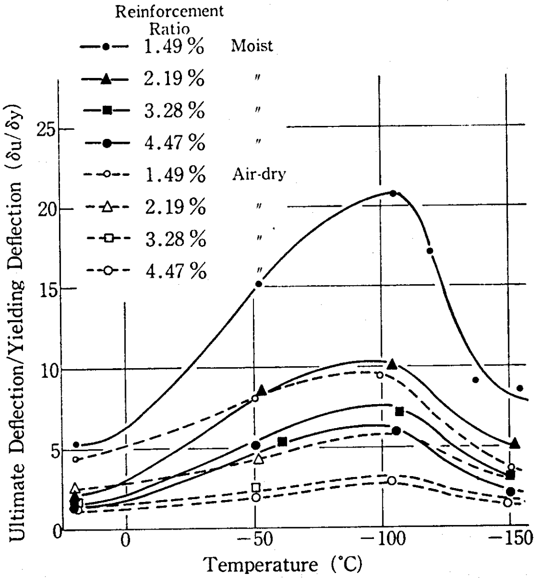


Fig. 29. Ductility Factor vs. Temperature

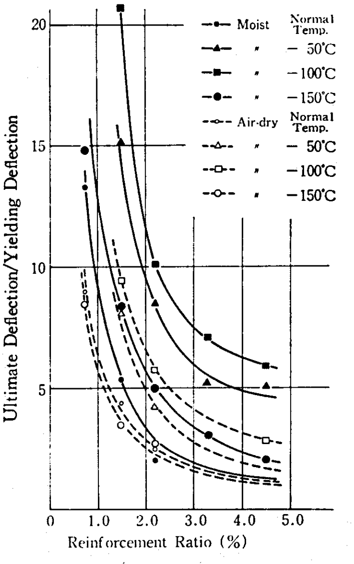


Fig. 30. Ductility Factor vs. Reinforcement Ratio

According to these results, the ductility factor of reinforced concrete is increased as temperature becomes lower, but when -100°C is passed it is conversely abruptly decreased, and at around -150°C it is lowered to the level at normal temperature, while especially, with specimens in air-dry condition, it is decreased even more. The cause of this is thought to be the change in the condition of ice inside the concrete at around this temperature. On comparing the ductility of reinforced concrete at low temperatures with it at normal temperatures, we find even though ductility factors are the same, beams at low temperature show sudden strength reductions after reaching maximum load. Therefore, when we consider ductility of a beam, it is not sufficient to make comparisons only by ductility factor, and it is thought the degree of strength reduction after reaching maximum load must be considered.

As one method of expressing this point, the deflection of the beam until bearing capacity became lower than 3/4 of yielding load was determined and a comparison was made with the value obtained by dividing this by the deflection of the beam at yielding. These relationships are shown in Fig. 31.

An example of methods of imparting ductility of the same degree as at normal temperature to reinforced concrete beams at low temperature using these results is indicated below.

In case of making reinforced concrete beams using concrete of σ_{ck}=370 kg/cm^2 and reinforcing steel of σ_{sy}=35.7 kg/cm^2 at normal temperature, when designing is done by the allowable stress design method with allowable stress as σ_{ck} /3 and σ_{sa} =2000 kg/cm^2, the reinforcement ratio will be about p=1.4%. According to Fig. 30, the ductility factor of a beam in air-dry condition is 4.35, and in order to obtain a corresponding ductility at -150°C, the reinforcement ratio must be made approximately 10% lower at p=1.25%. However, when abrupt reduction in bearing capacity is considered and Fig. 31. is used, the result will be

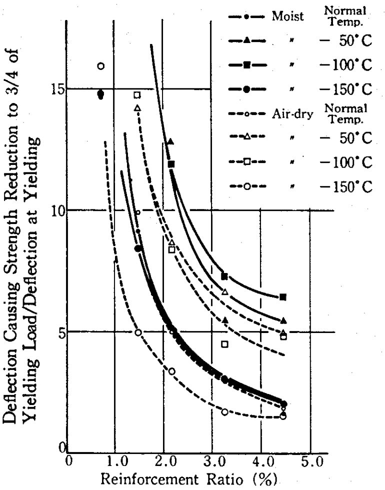


Fig. 31. Ductility Considering Beavior after Ultimate Deflection vs. Reinforcement Ratio

p=0.97%, requiring the reinforcement ratio to be reduced as much as approximately 30%. Although there is the possibility that a fair amount of error is included in the above calculations because of factors such as the accuracy of the diagram used, the trend will be as described above, and with regard to the ductility of a reinforced concrete beam, thorough attention must be paid to it since there will be cases where it will be considerably reduced at low temperatures under about -100°C.

Ductility factors of reinforced concrete members are often expressed by ϕ_u/ϕ_y based on the curvatures when the members are deformed on being subjected to bending moments in order that the ductility factors will be of general applicability. Here, ϕ_u is the curvature, as shown in Fig. 27, when ultimate moment (M_u) acts, and ϕ_y is the curvature of the beam when yielding moment (M_y) acts.

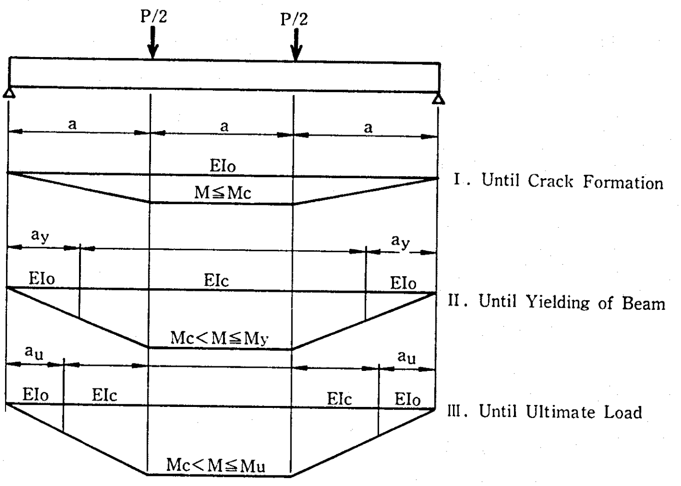


Fig. 32. Changes in Beam Stiffness according to Sizes of Loads

When we design a structure, it is generally considered necessary for this beam ductility factor to be about 4 to 6 [9], but it is extremely difficult to determine at what level it should be taken, and it will differ with requirements arising from hinge action of statically indeterminate structures, or requirements from energy absorption during earthquake action or the like, and otherwise, with the type and characters of the loads on the structure.

In order to be able to make comparisons with other materials, ductility factor based on curvature was determined from the results of the present experiments in the manner described below. It is thought there is some degree of error contained in this method, but calculations were made as one method of determining curvature from the load-deflection curve of a reinforced concrete beam.

Firstly, it is assumed that bending rigidity EI when a load acts on a reinforced concrete cross section may be divided into three stages: (1) before crack formation (EI_0) , (2) from crack formation to yielding of reinforcing steel (EI_c) , and (3) after yielding of reinforcing steel. And if it is assumed that yielding moment and ultimate moment are roughly proximate to each other, as indicated in Fig. 27. and 32, the condition of the beam may also be classified into three stages according to acting moment.

Therefore,

(1) Until crack formation

$$EI_0 = \frac{23P_c a^3}{48\delta_c} \tag{9}$$

(2) From crack formation to yielding of beam

$$EI_c = \frac{a^3 EI_0(23P_y^3 - 8P_c^3)}{48EI_0 P_y^2 \delta_y - 8a^3 P_c^3} \tag{10}$$

Hence, the curvature ϕ_y and curvature radius ρ_y of the bending span at yielding of the beam are

$$\phi_y = \frac{1}{\rho_y} = \frac{P_y a}{2EI_c} \tag{11}$$

(3) Until ultimate load

Since deflection after yielding of the beam is large, when solved using the relation $\frac{d^2y/dx^2}{\{1+(dy/dx)^2\}^{3/2}} = -\frac{1}{\rho_u}$, the result will be

$$\delta_u = \rho_u + \frac{P_c^3 a^3}{6EI_0 P_u^2} + \frac{a^3}{6EI_c P_u^2}(P_u^3 - P_c^3) + \frac{a^2}{\sqrt{4\rho_u^2 - a^2}} - \frac{\sqrt{4\rho_u^2 - a^2}}{2} \tag{12}$$

and the curvature radius ρ_u and curvature $\phi_u(=1/\rho_u)$ of the bending span at ultimate load are obtained.

Here

a : length of one third of span of beam

δ_c : deflection at middle of beam at time of crack formation

δ_y : deflection of middle of beam at yielding

δ_u : deflection of middle of beam at ultimate load

The results of calculations by this method[11] are shown by Figs.33. and 34, and it may be seen that trends roughly equal to those of Figs.29. and 30. are indicated.

Next, the strains ε_u at upper fibers of these reinforced concrete beams at maximum loads were compared.

The method of determining these strains was to divide x (Eq. (5), Ep. (7)), obtained when ultimate moment was calculated, by the curvature radius ρ_u (Eq. (12)) at ultimate load.

$$\varepsilon_u = \frac{x}{\rho_u} = x \cdot \phi_u \tag{13}$$

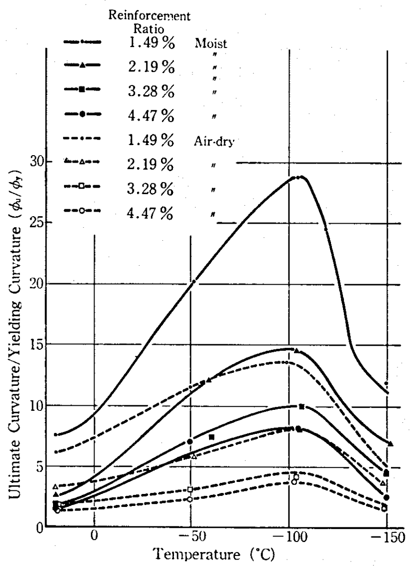


Fig. 33. Ductility Factor (ϕ_u/ϕ_y) vs. Temperature

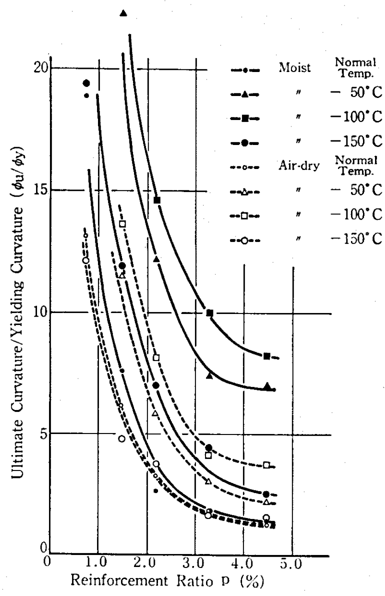


Fig. 34. Ductility Factor (ϕ_u/ϕ_y) vs. Reinforcement Ratio

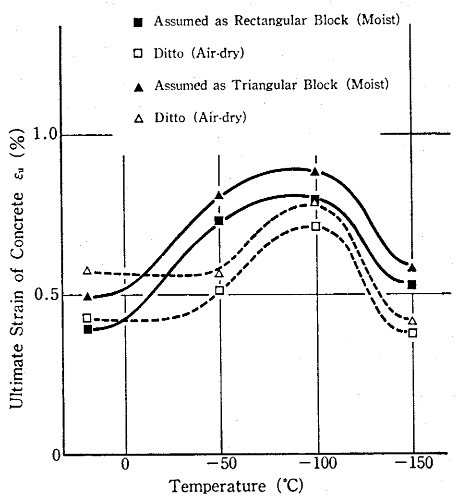


Fig. 35. Ultimate Strain of Concrete vs. Temperature

The results of these calculations are given in Fig. 35. It should be noted that when ultimate moments were calculated for Table 7, there were cases where reinforcing bar strains became excessively so large that it did not match calculated values, and for these, stresses of reinforcing bars were inversely calculated by Eq. (6) and Eq. (8) using measured values of ultimate moment, and x was calculated by employing these results. It is thought the accuracy of this method is fairly poor, and is only of such a degree as to provide a rough measure, but from the results, it may be seen that maximum strain of concrete also increases with temperature reduction, but below about -100°C it is conversely reduced.

In view of the above, it may be considered that one of the reasons why the ductility factor of concrete decreases at -150°C is the reduction in maximum strain of the concrete.

The experiments here were made on single reinforcement beams, whereas it is thought that the ductility of a concrete beam eill vary considerable with reinforcing bar arrangement, such as in case of double reinforcement or in case of using strrups. Consequently, it will be necessary to examine the influences of differences in bar arrangment in further studies, but it is thought the trend of change in properties of beams caused by lowering of temperature will be similar.

5. Conclusions

As a result of experimental studies of compressive and tensile strengths of concrete, characteristics of bond of concrete to deformed bars at low temperatures down to about -160°C, and of shrinkage strain of concrete when temperature is lowered, the conclusions below were drawn.

1) The compressive strength of concrete is generally increased when the temperature is lowered. The compressive strength σ_{cL} (kg/cm^2) at low temperature may be estimated by the equation below.

$$\sigma_{cL} = \Delta\sigma_c + \sigma_{c0}$$

$$\Delta\sigma_c = \left\{120 - \frac{1}{270}(T+180)^2\right\} \cdot w$$

$$\text{provided that } \Delta\sigma_c \leqq 107 \cdot w$$

where σ_{c0} is compressive strength (kg/cm^2) of concrete at a normal temperature, w is moisture content(%) of concrete, and T is temperature (°C) of concrete.

2) The tensile strength σ_T (kg/cm^2) of concrete at a low temperature may be estimated from compressive strength σ_c (kg/cm^2) by the equation below.

$$\sigma_T = 0.38\sigma_c^{3/4}$$

3) The maximum crack spacing of concrete near deformed bars at -160°C is 40% to 80% large than that at the normal temperature, and large than this in certain cases.

4) Anchorage strength to concrete and lapped splice strength of deformed bars at a low temperature are increased more or less proportionally to tensile strength of concrete at the low temperature, but the increase ratio is lowered in case of the latter when the length is increased.

5) The flexural strength of a reinforced concrete beam at very low temperature can be determined by calculations using the concrete strength and the reinforcing bar yield strength at that temperature.

6) The ductility of a reinforced concrete beam at flexure increases with lowering of temperature down to about -100°C, but abruptly decreases below that temperature and at -150°C it becomes about equal to the ductility at normal temperature. This is particularly affected greatly by the moisture content of the concrete, and a beam in air-dry condition will have considerably less ductility at -150°C than at normal temperature. Consequently, in designing reinforced concrete to be used under such a condition, the reduction in ductility should be considered, and in such cases it is thought one should take measures such as lowering reinforcement ratio.

References

1) Goto,Yukimasa, Takashi Miura and Yoshinori Abe: Properties of Concrete at Very Low Temperatures, Preprint of the 30th Annual Meeting of JSCE, No.5, pp.165-166, Oct 1975.
2) Miura,Takashi, Yoshinori Abe and Koji Kaminaga: Properties of Concrete at Very Low Temperatures, Abstracts, JSCE Tohoku Chapter Research Presentation Meeting, pp.223-224, March 1977.
3) Miura,Takashi, Yukimasa Goto and Yoshinori Abe: Relation between Moisture Content and Properties of Concrete that is used at Very Low Temperatures, Preprint of the 32nd Annual Meeting of JSCE, No.5, pp.46-47, Oct 1977.
4) Wischers, Gerd and J"urger Dahms: Das Varhelten des Beton bei Sehr niedrigen Temperature, Beton Harstellung Vermendung Jg.20, Heft 4, pp.135-139, April 1970.
5) Goto,Yukimasa and Takashi Miura: Mechanical Properties of Concrete at Very Low Temperatures, Proc. of 21st Japan Congress on Materials Research, pp. 157-159, March 1978.
6) Monfore,G.E. and A.E.Lentz: Physical Properties of Concrete at Very Low Temperatures, Jour. of P.C.A. Research and Development Lab. Vol.4, No.2, pp.33-39, May 1962.
7) Saito,Kimio,Toshio Hatsuzaki and Takashi Miura: Characteristics of Bond to Deformed Bars at Very Low Temperatures, Preprint of the 30th Annual Meeting of JSCE, No.5, pp.343-344, Oct 1975.
8) Goto,Yukimasa and Takashi Miura: Lap Splice and Bond behavior of Reinforcing Bars at Very Low Temperatures, Proc. of the JCI Symposium, pp.101-104, March 1976.
9) Ferguson,P.M.: Reinforced Concrete Fundamentals, John Wiley & Sons, p.325, 1973.
10) Miura,Takashi, Yoshinori Abe and Hideo Yoshizawa: Properties of Reinforced Concrete Members at Very Low Temperatures, Abstracts, JSCE Tohoku Chapter Research Presentation Meeting, pp.201-202, March 1977.
11) Miura,Takashi, Yoshinori Abe and Shigeru Kojima: Bending Strength and Ductility of Reinforced Concrete Beams at Very Low Temperatures, Preprint of the 33rd Annual Meeting of JSCE, No.5, pp.253-254, Sept. 1978.