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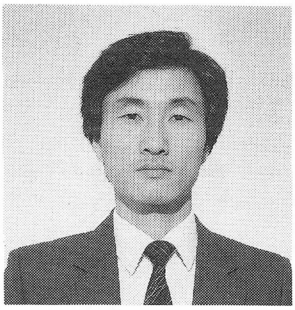
DEFORMATION AND DETERIORATION OF CONCRETE AT LOW TEMPERATURES

(Reprint from Proceedings of JSCE, No.420, 1990)



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SYNOPSIS

Under low temperatures, some concrete structures suffer serious deterioration. This is due to the ice formation in concrete micropores, which causes microcracking and loosens concrete microstructure. Based on this observation, we investigated deterioration of concrete subjected to cyclic cooling and heating of various temperature ranges. The two major findings are: (1) residual strain of concrete is in good correlation to its relative dynamic modulus of elasticity and (2) there are well-defined temperature ranges which govern the progress of deterioration. Qualitative and quantitative analyses are made on these observations.

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1. INTRODUCTION

Durability is one of the indispensable properties required for a concrete structure. However, concrete deteriorates under various severe environments, such as repetition of freezing and thawing. There are a number of concrete structures under environments which may cause serious deterioration.

In a cold region, a concrete structure is subjected to freezing and thawing. Even in a warm region, a particular concrete structure is used under low temperatures; for example, a storage tank of LNG (boiling point -162^{o}C) or LPG (boiling point -45^{o}C), a barge hull, or secondary structures to prevent spill from a primary tank. Recently, materials indicating superconductive properties near boiling point (-196^{o}C) of Nitrogen have been developed. Concrete will be applied at least, as a part of a structure which bears low temperature loads, such as a Superconductive Magnetic Energy Storage (SMET).

Concrete changes its mechanical properties in various aspects when it is cooled till very low temperature. It is known that the change is mainly due to the freezing of water in concrete; since phase transition from water to ice leads to about 9% expansion by volume, internal microcracks are initiated in concrete. And repetition of cooling and heating progress the deterioration of concrete, by continuous initiation or propagation of the cracks.

Deterioration of concrete is influenced by a range of temperature cycles, i.e. what temperature it is cooled and what temperature it is warmed. It is shown that although concrete can expand even under cooling, it shrinks almost linearly to temperature decrease below -50^{o}C; see [1],[2]. Hence, the temperatures which causes considerable deterioration in concrete is to lie between normal temperature and -50^{o}C.

Based on this observation, we investigated deformation and deterioration of concrete when it was subjected to cyclic cooling and heating of various temperature ranges between normal and -70^{o}C. All experiments were carried out in air such that water would not be absorbed into concrete specimens, and nondurable Non-AE concrete was used to accelerate deterioration.

Non-linear strain behavior under cooling may be viewed as a sign indicating deterioration, since it reflects internal expansion due to ice formation. Therefore, measuring strain behavior for cement paste, mortar, and aggregate under cooling and heating, and we monitored their deterioration such that their contribution to the concrete deteriorations could be clarified.

2. TEST PROCEDURE

In the present experiments, we used high early strength portland cement similar to ASTM Type III (specific gravity: 3.13), fine aggregate (specific gravity: 2.56), and crushed stone coarse aggregate (specific gravity: 2.87). Mix proportions of concrete are shown in Table 1, and mortar and paste used in the experiment have the same water cement ratio as concrete.

Figure 1 illustrates a typical experimental procedure. Figure 2 shows demensions of concrete, mortar, and paste specimens which were used for strain measurements. For measurement of temperature and strains, two thermocouples were mounted, one inside and the other on a surface, and strain gages were attached on opposite surfaces. For measurement of dynamic modulus of elasticity a specimen without thermocouples and strain gages was used.

Table 1 Mix proportions of concrete

Gmax (mm)	Slump (cm)	Air (%)	W/C (%)	s/a (%)	Unit Content (kg/m³)			
					W	C	S	G
25	11±1	2±0.5	56	42	194	346	725	1125

In order to avoid moisture losses during tests, all specimens were coated just after being taken out from a storage pool. The rate of moisture content change in concrete was observed to be less than 2% till 30th cycle, and hence the moisture loss during test was negligible.

A typical pattern of cooling and heating is shown in Figure 3. The maximum cooling and heating rate were 0.35°C/min, 0.90°C/min, respectively. During cooling, temperature difference between the surface and the inside was less than 5°C for concrete specimens, and 2°C for mortar or paste specimens.

Temperature of specimen was calculated as an average of those measured on the surface and inside (this treatment was verified from preliminary experiments which investigated internal temperature distribution of specimen). Measured

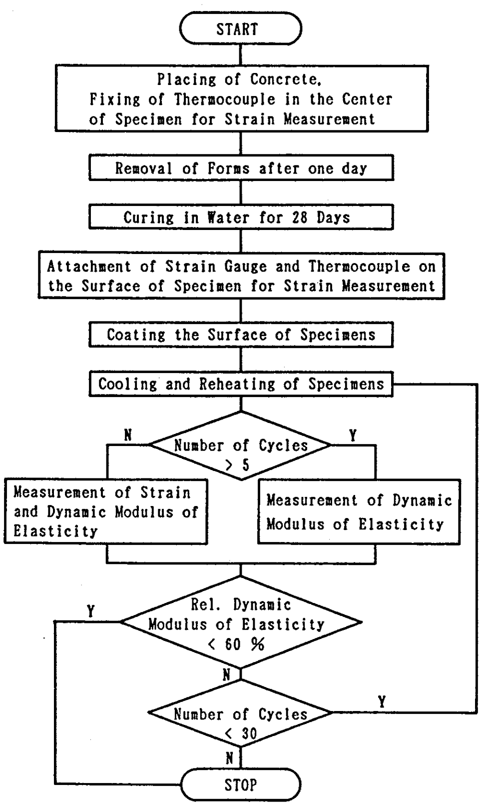


Fig.1 Flow chart of testing procedure

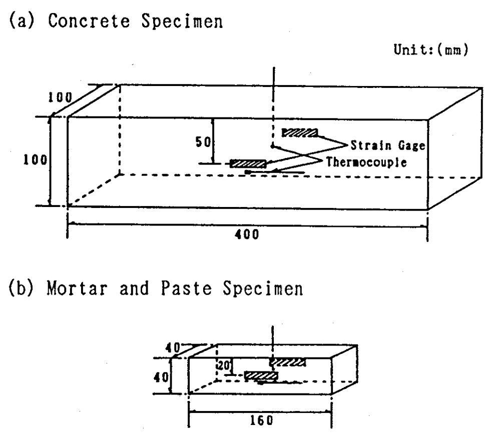


Fig.2 Prism specimens for strain measurement

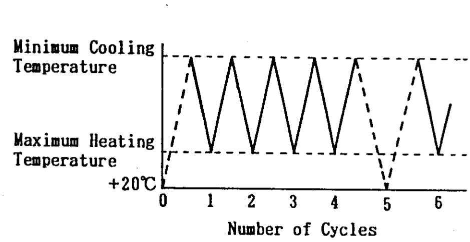


Fig.3 Temperature cycle for specimen

strain in the surface was regarded as average strain of the specimen. We considered a relation between the average strain and the average temperature.

Dynamic modulus of elasticity was measured after every five cycles and until 30th cycle. When relative dynamic modulus of elasticity was below 60%, the experiment was terminated.

3. RESULTS AND DISCUSSIONS

3.1 Theoretical background

If it is trapped in a micropores, water does not always freeze at $0^{\circ}C$. From measurement of enthalpy variation associated with cooling temperature on cement paste specimens, it is shown that free water freezes gradually as temperature goes down from $-4^{\circ}C$ to $-90^{\circ}C$ [3]. Depression of freezing point depends on the pore size, as shown in Figure 4 [4]; pore distribution in concrete used in this study is also presented. It should be noted that depression of freezing point is influenced by other factors, such as supercooling, chemical impurity in pore water, or surface energy interaction between water molecules and pore wall [5].

Although dried concrete exhibits a linear thermal deformation, water saturated concrete does a non-linear one. Figure 5 shows a typical strain behavior of water saturated concrete from normal to $-196^{\circ}C$. The concrete exhibits a non-linear thermal deformation, though it only contracts in a linear fashion below $-70^{\circ}C$ [6]. Hence, we can assume that temperatures below $-70^{\circ}C$ have no considerable influence on deterioration of concrete. Based on the above facts, this study restricts cyclic temperature ranges to lie between normal and $-70^{\circ}C$.

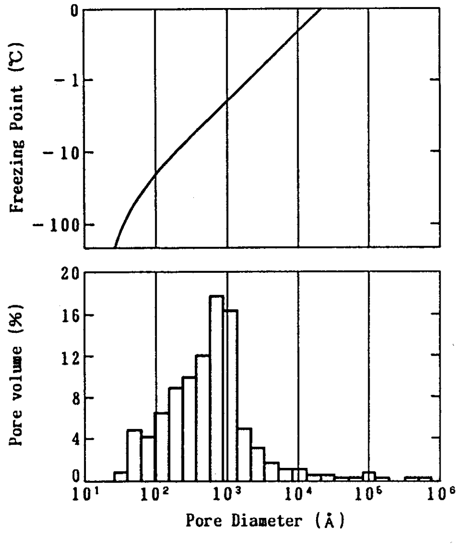


Fig.4 Relation between pore diameter and freezing point

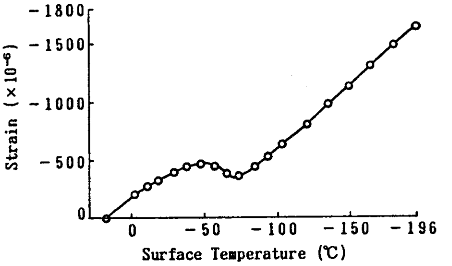


Fig.5 Typical relation between strain and surface temperature

3.2 Effect of the minimum cooling temperature

There are various temperature ranges to which a concrete structure can be subjected, and possible deterioration of concrete depends on the temperature range. To identify temperature ranges which cause serious deterioration, we

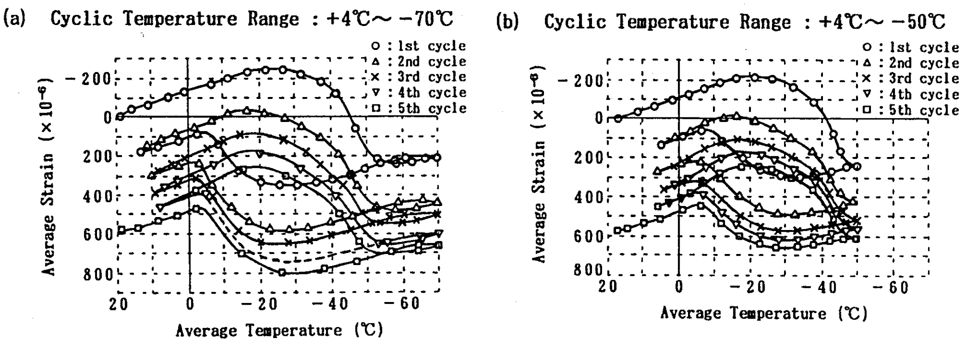


Fig.6 Relation between strain and temperature for various minimum cooling temperatures

carried out a series of experiments with varying the maximum or minimum temperatures. Figure 6 shows a typical strain behavior of water saturated Non-AE concrete under cooling and heating cycles. Major characters of the behavior are summarized [7],[8], as follows:

a) from +20°C to -20°C
Concrete shows a almost linear contraction. It seemed that most of the water in micropores might not freeze before -20°C, and that internal stresses caused by ice formation might be released, as the surplus of frozen water in larger pores transports toward smaller pores or partially empty voids pre-existing in concrete [9].

b) from -20°C to -50°C
Due to ice formation in almost all micropores, internal stresses become high and microcracks are initiated, showing considerable expansion of concrete. This expansion is closely related to deterioration. The amount of expansion increases as water cement ratio, moisture content, and cooling rate [7],[10-14] increases.

c) below -50°C
Since most of water has been already frozen, concrete exhibits a linear contraction again. However, absorbed water layers on walls of very small gel pores of radius less than 3 nm do not freeze within the range of 0°C to - 165°C [15].

d) on heating
Strain behavior during heating is not the reverse of that during cooling, since melting point of water in micropores differs from freezing point. There are created residual strains, when the temperature returns to normal. It is obvious that these residual strains are a sign that internal structure of concrete is loosened.

Since residual strains indicate an amount of microcracks generated in concrete, they can be a measure for deterioration. Figure 7 shows relationship between subtraction of the residual strain of the first cycle from that of each cycle and the number of cooling and heating cycles, for experiments with different minimum temperatures and fixed maximum temperature. The residual strains increase as the cooling process is repeated, though the amount of the increased residual strain diminishes. And the residual strains tend to increase as the minimum cooling temperature decreases. However, temperatures below -50°C do not

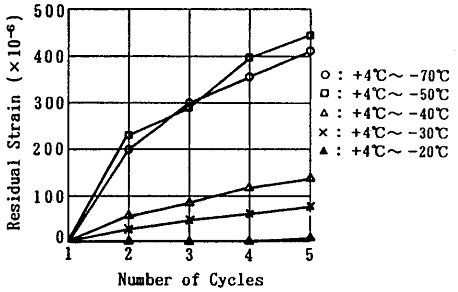


Fig.7 Relation between residual strain and number of cycle

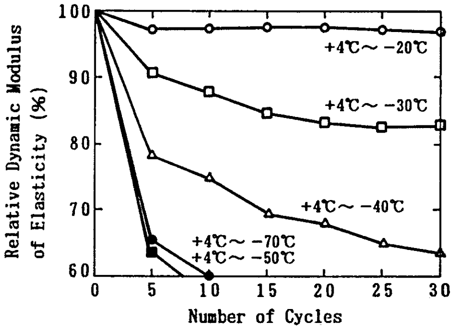


Fig.8 Deterioration of concrete due to repetition on cooling with fixed maximum temperature and various minimum temperature

cause extra deteriorations, since the residual strains in the case of the minimum temperature -70°C as those in the case of the minimum temperature -50°C are almost the same. Concrete can be hardly deteriorated when temperature is above -20°C, since the residual strain is not observed in the case of the minimum temperature -20°C.

Based on the above results, we may set the following three ranges for the minimum cooling temperature that determine a degree of deterioration:

i) above -20°C: concrete is hardly deteriorated
ii) -20°C ∿ -50°C: degree of deterioration increases as minimum cooling temperature decreases
iii) below -50°C: degree of deterioration is almost constant

We consider effects of the minimum temperature on the loss of dynamic modulus of elasticity. Figure 8 shows a change of the dynamic modulus of elasticity during repetition of cooling and heating when various minimum cooling temperatures are used. It is seen that concrete is hardly deteriorated for the minimum cooling temperature -20°C, since the loss of elasticity is very small. The relative dynamic modulus of elasticity at 30th cycle is about 80% and 60%, respectively, when the minimum temperature is -30°C and -40°C. In the cases of the minimum temperature -50°C and -70°C, the degrees of deterioration are almost the same, the relative dynamic modulus elasticity is less than 60% between 5th and 10th cycle.

As for effects of minimum cooling temperatures on deterioration, we can obtain a similar tendency in the results of the dynamic modulus of elasticity and the strain behavior. The mechanism of how the minimum cooling temperature effects on deterioration may be explained as follows: microcracking and non-linear thermal deformation of concrete are induced by about 9% volume expansion due to phase transformation of water, and the amount of ice formed in concrete increases as the temperature decreases. Hence, it can be concluded that the deterioration of concrete subjected to repetition of cooling and heating, mainly occurs at the temperatures between -20°C and -50°C.

3.3 Effect of the maximum heating temperature

Although a number of papers concerning to frost damage have been reported, studies for the influence of maximum heating temperature on deterioration of

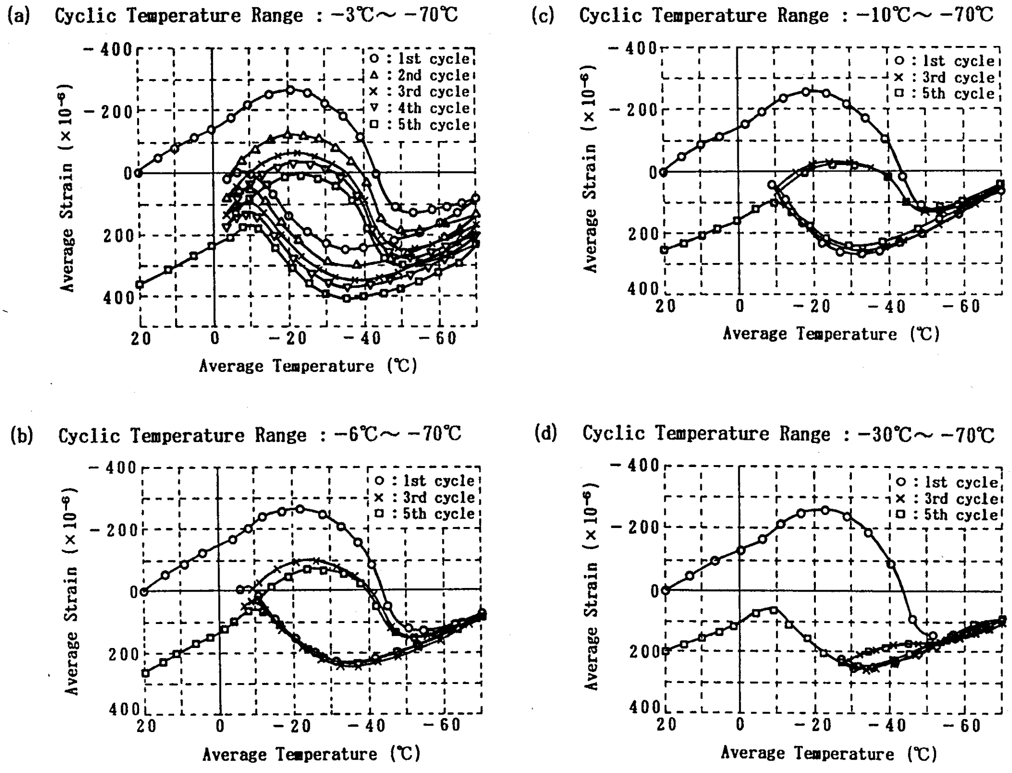


Fig.9 Relation between strain and temperature for various maximum heating temperature

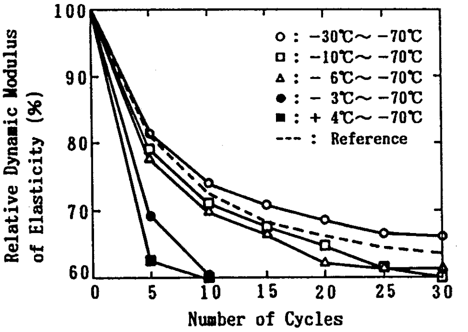


Fig.10 Deterioration concrete due to repetition of cooling with fixed minimum temperature and various maximum temperatures

concrete are still insufficient. We expect that the maximum heating temperature has a major influence of deterioration of concrete as well as the minimum cooling temperature. In this section, we investigate deterioration of concrete under temperature ranges with fixed minimum temperature and various maximum temperatures.

Figure 6 shows that concrete expands till -30°C during heating, and that it considerably contracts between -30°C and -10°C, and then expands again when the temperature is above -10°C. For cases, similar behaviors are observed; see Figures 9 where the minimum cooling temperature is -70°C and the maximum heating temperature ranges from

-30°C to +4°C. It is shown that strain behavior of concrete shifts downward as the temperature cycles proceed with the maximum heating temperature -3°C, and hence the residual strain increases. However, with the maximum temperature -6°C, -10°C, or -30°C, such shifts are not observed for all cycles. Hence, deteriorations of concrete depend on the maximum heating temperature in the range between about -10°C and zero.

Figure 10 shows change of dynamic modulus of elasticity during the repetition of cooling and heating when various maximum temperatures are used. Full lines in Figure 10 indicated influences of the designed temperature cycles as well as those of the initial and final cycles between +20°C and -70°C. A dotted line named for reference indicates the latter, it is a strain behavior of one cycle from +20°C and -70°C. Hence, we can approximately obtain the influences of the designed temperature cycles by taking the differences between the corresponding full line and the reference; namely, the full line lower than the reference suggests that the corresponding designed maximum temperature makes some contributions on deterioration of concrete, and the full line close to the reference suggests that the corresponding designed temperature makes little contributions on deterioration. Hence, the degree of deterioration is almost the same as the reference, when the maximum heating temperature is -30°C, -10°C, or -6°C. and it becomes greater than the reference when heating temperature is -3°C and +4°C. The result for loss in dynamic modulus of elasticity agrees well with that for residual strain studied in the above.

The above results may suggest that concrete is not deteriorated in some cases, even if it shows expansion during cooling. This is clearly seen from Figure 9(b); though concrete expands considerably from -20°C to -50°C on second cycle, the residual strain does not increase and the dynamic modulus of elasticity does not decrease, as the repetition of cooling and heating proceeds. The mechanism of this observation can be explained as follows: ice in pores is thawed from smaller pore as the temperature increases, and water is scarcely transportable due to the ice in larger pores which is not yet thawed till the temperature is up to -10°C. Therefore, it seems that deterioration can proceed if the temperature of concrete rises up till water in concrete is transportable and then water is frozen in other places during cooling. Hence, a non-linear thermal behavior of concrete only shows the phase transformation, does not indicate necessarily initiation of microcracks due to it.

From experiments for mortar [4],[10], the temperature at which mortar starts expansion during cooling or that at which mortar starts contraction during heating is almost the same, regardless of water cement ratio. Consequently, the transite temperature does not depend on the mix proportions of concrete, even though they change the porosities or pore distributions of concrete [9].

Therefore, it can be expected that the temperature ranges that influence deterioration of concrete may have little dependence on the mix proportions. The ranges identified in this study can be applicable to various concretes. However, the degree of deterioration is influenced by the pore distributions or porosities determined by the mix proportions, since the depression of freezing point is a function of pore size.

3.4 Strain behavior of constituent of concrete

Constituents of hardened concrete are pastes, sands, and coarse aggregates. Since global behaviors or properties of concrete are determined by those of the constituents, it is worth investigating how each constituent deteriorates under repetition of cooling and heating, in order to understand mechanism of concrete deterioration. Under the same conditions, a saturated specimens of aggregate, paste, mortar, and concrete are subjected to freezing and thawing, and strain behaviors are monitored; water cement ratio of paste, mortar, and concrete are the same. Since material may not be homogeneous from the top to the bottom due to material segregation, we assumed strains measured on the center of a specimen as an average strain of the specimen; see Figure 2.

Figure 11 shows strain behaviors of each constituent during cooling at first cycle. A thermal expansion coefficient of the constituents and the residual strains after the fifth cycle are calculated. Table 2 shows the maximum value of expansions. From Figure 11 and Table 2, the maximum expansions and the residual strains after the fifth cycle were larger, in the order of paste, mortar, and concrete, and aggregate which was not expanded at all. Hence, strain behavior and deterioration of concrete are mainly due to cement paste.

Since the difference in thermal expansion coefficients of cement paste and aggregate is relatively large, there is great possibility of interfacial fracture between cement paste and an aggregate, which may contribute deterioration of concrete.

Aggregate used in this study is of good quality, high density, and low porosity, and its strain behavior is almost linear, and i.e. shows scarce residual strain, when it is subjected repetition of cooling and heating. Hence, there should be little contribution from aggregate on deterioration of concrete.

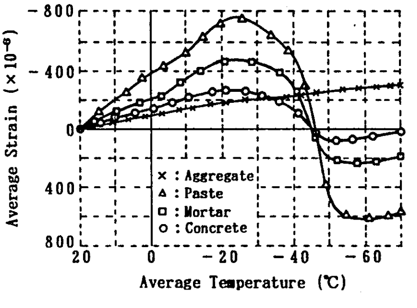


Fig.11 Relation between strain and temperature of concrete constituents

From the above results, it seems that deterioration of concrete is caused by the following two reasons: 1) deterioration of paste and 2) interfacial fracture between aggregate and paste. Hence, we can increase durability of concrete; by making durable cement paste of lower water cement ratio or lower moisture content, and AE agent [16].

Table 2 Properties of concrete constituents

Type of concrete constituent	Coefficient of thermal expansion ($\times 10^{-6}$/℃) -10℃	Coefficient of thermal expansion ($\times 10^{-6}$/℃) -45℃	Maximum expansion ($\times 10^{-6}$)	Residual strain after the 5th cycle ($\times 10^{-6}$)
Aggregate	4.6	3.0	——	0
Paste	17.5	−122.9	1367	908
Mortar	13.0	−66.9	703	582
Concrete	6.9	−25.7	376	445

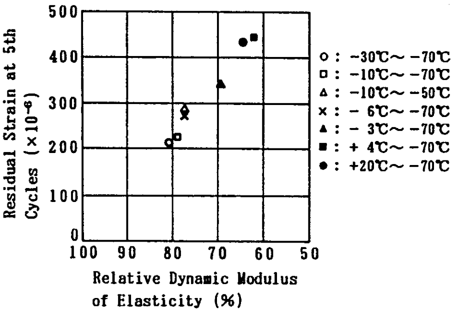


Fig.12 Correlation between residual strain and relative dynamic modulus of elasticity

3.5 Synthetical consideration

Figure 12 shows a relation between the relative dynamic modulus of elasticity and residual strain at fifth cycle for various concrete. It is seen that the residual strain and the relative dynamic modulus of elasticity are well correlated to each other [17],[18]. Consequently, the residual strain can be a good parameter for assessment of potential frost damage, which agrees with the result of MacInnis et al. [19].

Figure 13 shows a relation between the residual strain at fifth cycle and the minimum cooling temperature, and Figure 14 shows a relation between the relative dynamic modulus of elasticity and the minimum cooling temperature. It is seen that the degree of concrete deterioration is greatly changed, depending on the value of minimum cooling temperature lying between -20°C and -50°C. In particular, the range between -40°C and -50°C seems to have the greatest effects on deterioration.

Figure 15 shows a relation between the maximum heating temperature and the residual strain at fifth cycle, and Figure 16 shows a relation between the maximum heating temperature and the relative dynamic modulus of elasticity under

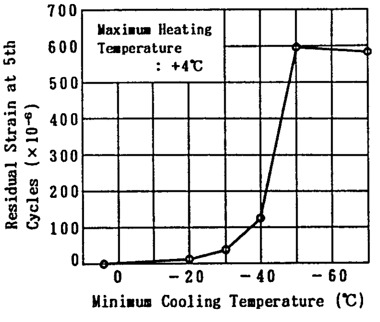


Fig.13 Relation between residual strain and minimum cooling temperature

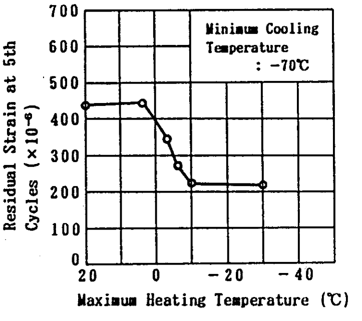


Fig.15 Relation between residual strain and maximum heating temperature

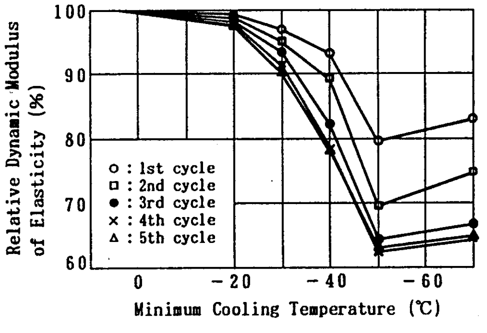


Fig.14 Relation between relative dynamic modulus of elasticity and minimum cooling temperature

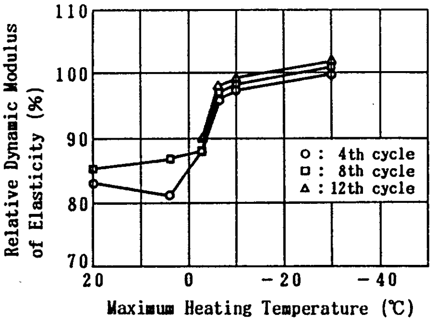
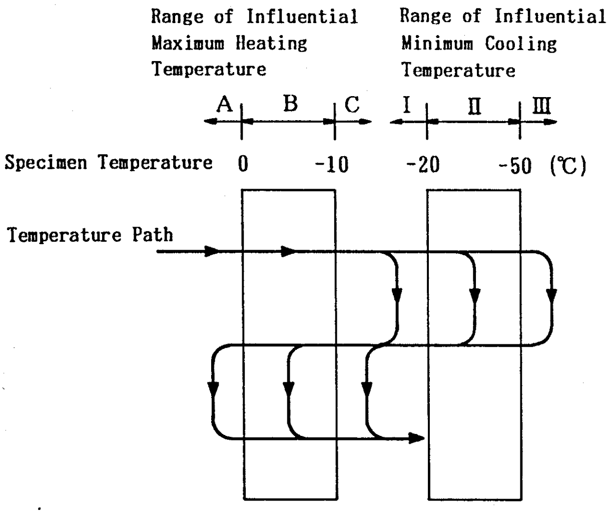


Fig.16 Relation between relative dynamic modulus of elasticity and maximum heating temperature



Degree of Concrete Deterioration

Range of Maximum Temperature \ Range of Minimum Temperature	I (over −20℃)	II (−20℃ ∼ −50℃)	III (below −50℃)
A (over 0℃)	○	△	×
B (0℃∼ −10℃)	○	△	△
C (below −10℃)	○	○	○

○ : no deterioration
△ : depend on the temperatures
× : maximum deterioration

Fig.17 A model for estimating concrete deterioration

the designed cycles (shown as full line in Figure 3). It is seen that the maximum heating temperature lying between -10°C and 0°C has a considerable effects on deterioration.

Figure 17 is chart from which the degree of concrete deterioration is estimated, when the structure is subjected to repetition of cooling and heating. Based on the results obtained, we divide the minimum cooling temperature and the maximum heating temperature into three ranges, I, II, III for the minimum temperature or A, B, C for the maximum temperature.

The effects of the minimum cooling temperature on deterioration of concrete are summarized as follows: 1) if the temperature is above -20°C (Range I), concrete does not deteriorate; 2) if the temperature is between -20°C and -50°C (Range II), deterioration increases as the temperature decreases; and 3) if the temperature is below -50°C (Range III), deterioration is almost the same as the case of the minimum temperature -50°C. The effects of the maximum heating temperature are summarized as follows: 1) if the temperature is below -10°C (Range C), concrete does not deteriorate even the minimum cooling temperature is low enough; 2) if the temperature is between -10°C and 0°C (Range B), deterioration increases as the temperature increases; and 3) if the temperature is above 0°C (Range A), deterioration is almost the same as the case of the maximum temperature 0°C.

Based on the above summaries, we can conclude that when the minimum cooling temperature or the maximum heating temperature is under Range I or C respectively, concrete suffers little deterioration; when the temperatures are under Range II and B, deterioration increases as the cooling temperature decreases or the heating temperature increases; but, when the temperatures are in Range III and A, concrete shows the greatest deterioration. However, Range B is very small, and the degree of deterioration changes significantly within Range B. Hence, from a practical point of view, it is better to estimate that deterioration occurs when the maximum heating temperature is above -10°C.

The above results suggest that no deterioration occurs in concrete, when it is not cooled down to -20°C or not heated up to -10°C in case of the cooling temperature was lower than -20°C. If these conditions are not satisfied, concrete can suffer deterioration when it is subjected to temperature cycle every time.

4. CONCLUSIONS

This study investigates how concrete deterioration is influenced by the cyclic temperature range, and what constituent of concrete has a contribution to deterioration, as concrete is exposed to low temperatures. Based on the experimental results, the following conclusions can be drawn:

(1) Degree of concrete deterioration depends on the minimum cooling temperature and the maximum heating temperature of cyclic temperature range.

a) When the minimum temperature is above -20°C, concrete suffers little deterioration; degree of deterioration increases as the minimum temperature dncreases from -20°C to -50°C, and no further deterioration occurs even when the temperature goes down below -50°C.

b) When the maximum temperature is below -10°C, concrete does not deteriorate; degree of deterioration increases as the maximum temperature increases from -10°C to 0°C, and no further deterioration occurs even when the temperature goes up above 0°C.

Hence, in practical structures, if the temperature of concrete falls below -20°C and rises above 0°C, it is necessary to take a consideration to prevent deterioration due to each cyclic temperature ranges. Rational measures should be studied in future.

(2) Deterioration of concrete considerably depends on cement paste and interfacial fracture. Hence, cement paste should be made to be durable in order to produce a durable concrete, and interfacial fracture of the paste-aggregate should be paid sufficient attentions in studying deterioration.

(3) Loss of relative dynamic modulus of elasticity is almost proportional to residual strains. Both of them can be used as a good parameter for assessment of potential frost damage.

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