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EFFECT OF TEMPERATURE AND AGE ON THERMAL EXPANSION AND MODULUS OF ELASTICITY OF CONCRETE

BY
CARL BERWANGER
A. FARUQUE SARKAR

Synopsis: The thermal coefficient of expansion and the dynamic modulus of elasticity of concrete have been determined under short-term steady state temperatures, -100 to 150°F (-73 to 66°C). Specimens were cured both saturated and air-dried in the laboratory and tested at 7, 28, 84 days and at one year.

Thirty-five $3 \times 4 \times 12$ -in ($7.6 \times 10 \times 30.5$ -cm) concrete prisms were tested for thermal coefficient of expansion. The coefficient of expansion decreased with increase in the water-cement ratio, increased with age and was smaller below the freezing point of the concrete. The saturated concrete had lower coefficients of expansion than the air-dried concrete.

Thirty-six $3 \times 3 \times 12$ -in ($7.6 \times 7.6 \times 30.5$ -cm) concrete prisms were used to determine the dynamic modulus of elasticity. The dynamic modulus of the saturated concrete increased while that of the air-dried concrete decreased with age, under normal laboratory temperatures. The modulus also increased with decrease in the water-cement ratio and with temperature decrease. There were greater increases in the dynamic modulus below the freezing point, which also decreased with age. Eighty-four 6×12 -in (15.2×30.5 -cm) control cylinders were tested for compressive strength and the static modulus of elasticity of concrete.

Keywords: age; aggregates; compressive strength; concretes; dynamic modulus of elasticity; low temperature; modulus of elasticity; strains; thermal expansion.

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Carl Berwanger, ACI member, is professor of civil engineering, University of Ottawa, Canada. He is a past-president, Canadian Capital Chapter of ACI, member of ACI-ASCE Committee 428, Limit Design, and chairman subcommittee No. 5, computer documentation, ACI Committee 118, Use of Computers.

A. Faruque Sarkar is a research associate in civil engineering, University of Ottawa, where he received his M.A.Sc. degree in 1972. He is a B.Sc. graduate of an engineering university in Bangladesh.

INTRODUCTION

Concrete is normally reinforced with steel; deformed steel bars and prestressing wires are embedded and bonded to the concrete, and prestressing tendons are also used. These produce the common composite members of reinforced and prestressed concrete structures. Composite members are also formed from steel beams with reinforced concrete slabs connected to their top flanges. Whether through the action of bond or by means of shear connectors, these two materials, which have different thermal coefficients of expansion, are forced to deform together producing stresses when exposed to temperature change. The importance of the problem can be seen in the extensive use of composite construction for highway bridges and the increased trend to make use of architecturally exposed structural frames in buildings. These exposed structures would require analyses for temperature stresses in order to adequately complete their design. A knowledge of the thermal properties of these materials is essential for an accurate analysis of exposed structures for thermally induced stresses.

The average thermal coefficient of linear expansion of steel at normal temperatures is quoted as, $K_s = 0.0000065/^{\circ}\text{F}$ ($0.0000117/^{\circ}\text{C}$) (1). The coefficient is assumed to increase linearly with increase in temperature to 1200°F (649°C), where it has a value of $0.0000084/^{\circ}\text{F}$ ($0.0000151/^{\circ}\text{C}$). No design values for the coefficient are quoted for below normal temperatures. The important temperature range for exposed structures, in Canada and the northern parts of the United States, is approximately -50° to 100°F (-46° to 38°C). In the investigations by Day and Sevand (2) and Majors (3), it was shown that the Young's modulus of steel (E_s) increased with a decrease in temperature. The variation was linear between 80° (27°C) and about -60°F (-51°C), where a non-linear trend started. The average modulus was 29,000 ksi ($20,400 \text{ kg/mm}^2$), with a total increase of 500 ksi (350 kg/mm^2) over this temperature range. The modulus increased more rapidly below -60°F (-51°C) reaching 31,000 ksi ($21,800 \text{ kg/mm}^2$) at -280°F (-173°C). The total variation in the modulus, over the temperature range for exposed structures, is less than two percent. It would seem that the average modulus could be used without significant error. Similarly, it is expected that the coefficient of linear expansion does not vary significantly over the temperature range for exposed structures. The

average values of $K_s = 0.0000065/^{\circ}\text{K}$ ($0.0000117/^{\circ}\text{C}$) and $E_s = 29,000$ ksi ($20,400$ kg/mm²) have normally been used in calculations for thermal stresses in composite members.

In the past, it was normally assumed that concrete had the same coefficient of expansion as its aggregates. Siliceous sand, a common component of concrete, has a coefficient that is very close to that of cement, $0.000007/^{\circ}\text{F}$ ($0.0000126/^{\circ}\text{C}$), and is also close to that of steel. This led to the conclusion that no thermal incompatibilities existed; in concrete between the cement paste and the aggregates and in reinforced concrete between the concrete and the steel reinforcement.

Some investigations into the durability of concrete included the measurements of the thermal coefficients of various common mineral aggregate used in concrete. The mean linear thermal coefficients of various natural aggregates varied from 0.000003 to $0.0000071/^{\circ}\text{F}$ (0.0000054 to $0.0000128/^{\circ}\text{C}$) in the investigation by Verbeck and Hass (4). Callan (5) reported coefficients for coarse aggregates that varied from 0.0000019 to $0.0000054/^{\circ}\text{F}$ (0.0000034 to $0.0000097/^{\circ}\text{C}$). Mortars made from these crushed aggregates had larger coefficients, 0.0000044 to $0.000007/^{\circ}\text{F}$ (0.0000079 to $0.0000126/^{\circ}\text{C}$), reflecting the influence of the cement paste. The temperature ranges for the test results quoted were for Verbeck and Hass, 77 to 84°F (25 to 29°C), and for Callan, 35 to 135°F (2 to 57°C).

A further review of the literature (6-9) reveals that there are thermal incompatibilities between the aggregate and the cement paste in concrete. The cement paste has an average thermal coefficient of expansion of about $0.0000074/^{\circ}\text{F}$ ($0.0000133/^{\circ}\text{C}$). Limestone aggregates show the largest deviations with a representative mean of $0.0000034/^{\circ}\text{F}$ ($0.0000061/^{\circ}\text{C}$), while quartz aggregates have the smallest deviations, with a mean of $0.0000054/^{\circ}\text{F}$ ($0.0000097/^{\circ}\text{C}$). Wide variations in the coefficient have been reported for various mineral types depending on the source and the impurities present. The mix proportions will greatly influence the coefficient of the concrete, the factors being the amount of aggregate and the water-cement ratio. Curing, that is, the amount of moisture present in the concrete greatly influences the thermal properties of concrete. Both the modulus and the coefficient were found to decrease with the loss of moisture and with increased age, which is usually accompanied by moisture loss through hydration and evaporation. The modulus of the concrete also increased with temperature decrease when moisture was present.

The first investigation to determine the coefficient of expansion of reinforced concrete slabs was reported by Cran (10) and Berwanger (11). Full-size slabs of plain and reinforced concrete were subjected to temperatures varying from -30 to 75°F (-34 to 24°). The dynamic modulus of elasticity was also measured, using $3 \times 3 \times 12$ -in. ($7.6 \times 7.6 \times 30.5$ -cm) concrete prisms, for the same temperature range.

Additional research on the coefficient of expansion of concrete has been reported by Walker, et al, (12) and a summary of the work in Germany has been reported by Dettling (13). The modulus of elasticity of concrete was reported for concretes undergoing low temperatures, 14

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to -76°F (-10 to -60°C), in references (14) and (15), and for above normal temperatures in references (16) and (17).

The investigation reported in this paper presents values for the coefficient of expansion and the dynamic modulus of elasticity of concrete. The test temperatures varied from -100 to 150°F (-73 to 66°C). The other variables were the water-cement ratio, 0.43 to 0.71; the age, 7, 28, 84 days, and one year; and the method of curing, water saturated and air-cured. The objective was to more completely define the coefficient of expansion and the modulus of concrete for the temperature range of exposed structures, including the important variables noted, while using a concrete whose aggregate mineral content reflected that found in normal construction.

EXPERIMENTAL PROGRAM

Specimens

Thirty-five 3 x 4 x 12-in (7.6 x 10 x 30.5-cm) concrete prisms were tested for thermal coefficient of expansion, thirty-six 3 x 3 x 12 in (7.6 x 7.6 x 30.5-cm) concrete prisms were used to determine the dynamic modulus of elasticity, and eighty-four 6 x 12-in (15.2 x 30.5-cm) control cylinders were tested for compressive strength and static modulus of elasticity of concrete. Temperatures varied from -100 to 150°F (-73 to 66°C).

The specimens were divided into eighteen series, each series comprised two of each type of concrete prism and five control cylinders. Ten series were dry-cured in the laboratory; 68 to 72°F (20 to 22°C) and 65 to 75 per cent RH; after an initial 7 days curing in water at about 70°F (21°C). Two series each having water-cement ratios of 0.45, 0.49, 0.58, and 0.67, and one series each having water-cement ratios of 0.43 and 0.71 were cast. The eight series cured in water until tested consisted of two series each having water-cement ratios of 0.45, 0.49, 0.58, and 0.67. The test ages were 7, 28, and 84 days for the specimens of water-cement ratios of 0.67, 0.58, and 0.49; 7 and 84 days for W/C = 0.45; and at one year for those with W/C = 0.43 and 0.71.

The cement used was Type 1 portland (ASTM C150-71) (18) purchased in a single lot from a local supplier. The aggregates were sieved into fractions and recombined to meet the mean grading curve of the limits of ASTM C 33-71 (19). The fine aggregates had a fineness modulus of 2.7. The maximum aggregate size was 3/4-in. (19 mm). The petrographic analysis indicated that the coarse aggregate was predominantly limestone (65%) with about 25% feldspar, 5% quartz, and 3% mica. The fine aggregates contained mainly quartz (35%), carbonates (29%), and feldspar (23%), with small amounts of amphibole, garnet, magnetite, mica and pyroxene. The concrete mixes had water-cement ratios from 0.43 to 0.71. The concrete strengths varied from 4800 to 6340 psi (337 to 446 kg/cm²) at 28 days.

Environmental Chamber

A specially constructed environmental chamber shown in Figure 1

was used for the temperature tests. The chamber operates through input controllers which operate motor-driven cycling devices that modulate the cooling and heating capacities. The Honeywell controller regulates temperatures through the range from -100 to $200 \pm 1^{\circ}\text{F}$ (-73 to $93 \pm 0.6^{\circ}\text{C}$).

Electrodynamic Materials Tester

The SCT-5 electrodynamic materials tester, shown in Figure 2, was used to determine the dynamic modulus of elasticity of concrete from $3 \times 3 \times 12$ -in. ($7.6 \times 7.6 \times 30.5$ -cm.) prism specimens. The prism is excited in the longitudinal mode and the vibrations are transmitted as voltages by the piezo-electrical crystal in the pick-up which activates the amplitude indicating meter.

To determine the dynamic modulus of elasticity (E_{cd}) of the prism material the resonant or natural frequency (n) is required. These quantities are related in the expression: $E_{cd} = 0.01035 (L/bt)Wn^2$. The resonant frequency method has been standardized in ASTM C 215 (20); where L = the length, b = the width, and t = the thickness of the prism, all in inches, and W = the weight of the prism in pounds.

The procedures for obtaining the static Young's modulus of concrete (E_c) from the compression test of standard cylinders are described in ASTM C 469 (21). This method results in the calculation of a chord modulus of elasticity. Other computations are possible giving the static modulus as the initial tangent modulus or the secant modulus as related to a particular stress level. It has been shown by Philileo (22) that the dynamic modulus is very close to the initial tangent modulus. In tests carried out at the University of Ottawa to correlate the static modulus, as determined from ASTM C 469, with the dynamic modulus, as measured with the SCT-5 and using ASTM C 215, the relation has been found to be: $E_{cd} = 1.14 E_c + 0.48 \times 10^6$, for the materials and the test procedures under comparison.

Optical Extensometers

The optical extensometer telescope assembly, shown in Figure 3, was used to measure the thermal coefficients of expansion of the concrete prisms. The specimens were arranged vertically on the chamber platform, as shown in Figure 4. The targets were made of crossed 0.001 in. (0.0024 -cm.) diameter Advance wires mounted on each of the 4 inch (10.2 -cm.) wide faces of the prisms at 10 in. (25.4 -cm.) centres.

Transducers

In addition to the optical extensometers, direct current displacement transformers (DCDT), were used to measure the deformations in two of the concrete prisms in each series and also of the aluminum deformation correction plates. These specimens are shown in Figure 4 at the right side of the prisms.

To check for errors in the optical extensometer readings, that might occur due to the frequent handling and resetting, two aluminum plates were used as controls for the deformation measurements, as shown

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in Figure 5. The aluminum plates had known deformation characteristics and had both targets for the optical extensometers and the DCDT's mounted on them. Any discrepancies could readily be detected by these control readings and the corrections applied to the readings.

PROCEDURES

The concrete was mixed in a 3.5 cu. ft. (p.1 cu. m.) capacity Eirich counter-rotary mixer and consolidated in steel moulds by means of a pencil vibrator. All specimens were cured under wet burlap for about 16 hours, demoulded, and then stored in water. The specimens to be dry-cured were removed from the water at seven days when the first measurements were also made.

The day before measurements were to be made, the wet-cured specimens were removed and allowed to become surface dry. The 4-in. (10.2-cm.) wide prism faces were marked locating the target positions using an invar scale pointer to give a 10 inch (25.4 cm.) gauge length. The Advance wires were attached by means of epoxy adhesive to form the extensometer targets. The aluminum blocks for the DCDT's were also mounted on two 4-in. (10.2-cm.) prisms in each series. The gauge length used for the DCDT's was 4 inches (10.2 cm.) , this left a clear line of sight for the extensometers. The prisms were placed on end on the steel platform in the chamber and plumbed vertically in order that the measured faces would be perpendicular to the line of sight of the extensometers (Figure 4). Thermocouples were embedded in some of the wet- and dry-cured specimens to measure the actual temperatures during the test. The lead wires for the thermocouples, the DCDT's, and the SCT-5 were brought through sealed chamber wall openings and connected to the appropriate instruments. The chamber doors were locked during the test period to insure that there would be no variation in specimen temperatures (Figure 1).

Control readings were taken at room temperature to start the test. Temperatures were decreased in 15°F (8°C) intervals and maintained at each temperature level until steady state conditions resulted. The thermocouple readings were checked from time to time to verify steady state conditions. At each temperature setting, readings were taken with the extensometers on 4-in. (10.2-cm.) prisms, the resonant frequency was determined for the 3-in. (7.6-cm.) prisms, and temperatures of the specimens were taken from the thermocouples. Temperatures were decreased in 15°F (8°C) intervals to -100°F (-73°C) then increased to 150°F (66°C) and finally decreased to room temperature (70°F or 21°C).

One of the control cylinders was tested at seven days, and two at each of 28, 84 days and 365 days to determine the static modulus of elasticity, using the method of ASTM C 469 (21), and the compressive strength of the concrete. The cylinders underwent the same curing as the prisms. The cylinders for test at later ages were placed at the bottom of the chamber (Figure 4) during the temperature tests.

EXPERIMENTAL RESULTS

Modulus of Concrete

The results of the cylinder tests are presented in Figure 6. The influence of the water-cement ratio, the age, and the method of curing on the static modulus of the concrete can be noted. Each plotted point is the mean of four cylinders. The static modulus increases with age. There is a greater increase for the wet-cured than for the dry-cured at a given age and water-cement ratio. The trend is linear between 0.49 and 0.67 for ages of 28 and 84 days, with a sharp increase for lower water-cement ratios indicated. The 28 day values for 0.45 were not available due to a break-down in the operation of the chamber. The 7-day results indicate a non-linear trend, with some of the modulus values at the lower water-cement ratios exceeding their 28-day results. The range of deviations from the plotted means is about 8 per cent for the 7-day results, about 4 per cent for 28-day results, and for all ages it is about 6 per cent. The 7-day results would seem to have greater variability and experimental error.

For comparison, the results of the prism tests are presented in Figure 7. The dynamic modulus is seen to decrease with the age of the laboratory dried prisms and increase with the age of the water-cured prisms. The dynamic modulus generally increases with a decrease in the water-cement ratio. The trend is linear for the 28- and 84-day wet-cured prisms, while it is non-linear at 7 days and also for the air-cured specimens. The deviations from the plotted means is generally less than 3 per cent.

In Figure 8, the ratio E_{cd}/E_c is plotted for common values of water-cement ratio, age, and curing. The general trend is for the ratio to increase with an increase in the water-cement ratio, and decrease with age. The 7-day values show a sharp increase and then a levelling-off after a $W/C = 0.6$. The 28-day results indicate linear and almost identical behaviour for the wet- and dry-cured specimens. This relation is approximately: $E_{cd}/E_c = 0.95 + 0.333 (W/C)$. After 28 days, the dry-cured specimens show the greater decrease in the ratio with an approximate linear relation at 84 days. The 84-day wet-cured specimens show the ratio levelling-off at about 0.5 after a sharp increase from the lower water-cement ratios.

The variation of the dynamic modulus over the temperature range -100 to 150°F (-73 to 66°C) is plotted in Figure 9 for a typical prism specimen. The general trend is for the dynamic modulus to increase with temperature decrease. Straight lines were fitted to the experimental points by means of the least squares method. The slopes give the rates of change of the dynamic modulus for the lines above and below the freezing point. There is a sharp increase below the freezing point produced by the frozen water present. The greater the amount of frozen water present the greater the slope below the freezing point.

Test results reported by Verbeck and Klieger (23) and also by Monfore and Lentz (9) show similar relationships for the modulus of the

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concrete, the water-cement ratio, and the amount of water frozen as the temperature decreased. The results reported by Berwanger (11) also showed slightly increasing modulus values from room temperatures to the freezing point with a sharp increase below this point.

Monfore and Lentz (9) reported an investigation that included the modulus of elasticity and thermal contractions of three sand and gravel concretes at temperatures from 75 to -250°F (24 to -157°C). The moisture present in the concrete during the test greatly influenced the properties. The modulus of elasticity of concrete with a cement content of 517 lb/cu. yd. (307 kg./cu. m.); when moist cured (28 days fog and 63 days sealed) increased from 5,500 to 8,500 ksi (3870 to 5980 kg/mm²) for temperatures from 75 to -250°F (24 to -157°C). The same concrete air-cured (28 days fog, 40 days sealed, and 80 days at 50% RH) increased from 5,000 to 5,400 ksi (3520 to 3800 kg/mm²) for the same temperatures. When the concrete was dry-cured (28 days fog, 94 days sealed, 35 days at 50% RH, and then 10 days at 221°F (105°C)) the modulus value was constant at 4,300 ksi (3020 kg/mm²) over the temperature range. It can be noted that the modulus of elasticity decreased with a reduction of moisture in the specimen, and also with its age.

Verbeck and Klieger (23) showed that as the temperature of concrete decreased below 32°F (0°C), the amount of water which is frozen increases. The ice that has been formed contracts upon further cooling, but the additional ice formed during this further cooling causes further expansion. The sharp increase in the modulus below freezing is produced by the frozen water. The larger the amount of frozen water present the greater the increase in the modulus.

The aggregates used in the concrete mixes of the investigation reported by Berwanger (11) were composed of about 75 per cent limestone with the remainder granite and granitoid gneiss and other metamorphic rock types. The fine material was predominantly limestone with some quartz, feldspar, and hornblende.

The compressive strength of the concrete varied from 2400 to 4800 psi (169 to 337 kg/cm²). The modulus increased slightly between room temperatures and the freezing point, 80 to 32°F (27 to 0°C), where there was a sharp increase in the modulus with further decrease in temperature to -40°F. The modulus increased with a decrease in the water-cement ratio and decreased with the age of the concrete. The slope below the freezing point was greatest at the earliest ages and with the largest water-cement ratios, that is, with the largest moisture contents in the specimens. The slope below freezing decreased with age, -0.0161 to -0.0012 x 10⁶ psi/°F (-0.00203 to -0.00015 x 10³ kg/cm²), for ages from 42 to 761 days respectively. The specimens were fog cured for 28 days and then stored in the laboratory.

A summary of the slopes (dE_{cd}/dT) for the test results of the prisms of this investigation have been plotted in Figure 10, for the dry-cured specimens and in Figure 11, for the wet-cured specimens. Each plotted point is the mean of four prism tests. It can be seen that the above freezing slopes decrease slightly with age for the dry-cured specimens, while there is a slight increase with age for the

wet-cured specimens. There is a general decrease in the slopes, both above and below the freezing point, with decrease in the water-cement ratio for both types of curing. The below freezing slopes show a very sharp decrease with age for the dry-cured specimens, while the wet-cured specimens show a more moderate decrease with age. The average range of deviations from the plotted mean results is 7 per cent in the case of the above freezing slopes and 6 per cent for the below freezing slopes.

The dry-cured specimens exposed to the laboratory air would have moisture removed through hydration and evaporation. There would be very little moisture absorbed by the concrete due to the relative dry laboratory air. The wet-cured specimens have water present during curing permitting hydration to proceed. The above freezing slope increase for the wet-cured concrete would reflect the increase in strength with age. The below freezing slope decrease would result from the advanced stage of the hydration process. This decreases the freezing point further leaving somewhat less absorbed water to freeze in the specimen and a higher concentration of the hydration salts.

Thermal Coefficient of Expansion

Figure 12 shows typical deformation-temperature results for one face of a concrete specimen, $W/C = 0.49$, at 28 days that had undergone curing in laboratory air. The experimental points are compared to straight lines which were determined separately for above and below freezing temperature values by means of least square fits. The slope values indicate a steeper line for the above freezing behaviour than for the below freezing deformations. The coefficients of correlation for these fits varied generally from 0.80 to 0.97 indicating very good correlation between the experimental points and fitted curve. The slopes are the coefficients of expansion for above and below freezing temperatures.

Figure 13 presents the results of these coefficients of expansion for above freezing temperatures, while Figure 14 indicates the coefficients for below freezing temperatures for all of the concrete specimens tested. Each experimental point is the mean of four slope determinations made on two concrete specimens. For above freezing temperatures; the coefficient of expansion is seen to increase in value with age for both the dry- and wet-cured specimens. The dry-cured specimens have the greater increase in the coefficient with age, for a given water-cement ratio. The coefficient increases with a decrease in the water-cement ratio. For below freezing temperatures; the trends are very similar to those for above freezing temperatures except that the increase with age is somewhat greater than that indicated for above freezing temperatures.

Verbeck and Klieger (23) also reported that a linear relationship existed between temperature and the concrete strains down to the freezing of water present in concrete. Below this freezing point the strains were found to be smaller than those that could be determined from a continuation of the above freezing straight line due to expansion caused by the freezing of water within the specimen. A maximum

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reduction of 0.000035 in. (0.00089 mm) was reported at -20°F (-29°) and at an age of 8 days. The deviation from the above freezing line would be somewhat less at greater ages. The amount of water available to be frozen at later ages would be reduced due to hydration and also by evaporation in the case of the air-dried specimens.

Mitchell (6) included concretes, neat cements, as well as aggregates in his thermal expansion tests. He also reported wide variations in the coefficients of aggregates depending on the source and the impurities present; for limestone from 0.0000014 to 0.0000065/ $^{\circ}\text{F}$ (0.0000025 to 0.0000117/ $^{\circ}\text{C}$) and for sandstone from 0.0000034 to 0.0000065/ $^{\circ}\text{F}$ (0.0000061 to 0.0000117/ $^{\circ}\text{C}$), were some of the 21 types of mineral aggregates investigated. The coefficient of neat cement varied from 0.0000046 to 0.0000121/ $^{\circ}\text{F}$ (0.0000088 to 0.0000218/ $^{\circ}\text{C}$) with a mean of 0.0000074/ $^{\circ}\text{F}$ (0.0000133/ $^{\circ}\text{C}$); variations in cement source and water-cement ratio were included. The mortars showed values of the coefficient from 0.000004 to 0.000007/ $^{\circ}\text{F}$ (0.0000072 to 0.0000126/ $^{\circ}\text{C}$) with limestone sands at the lower end and quartz sands at upper end of the range. For the concretes, the combination of limestone sands with limestone gravels gave the lowest coefficients, average value of 0.0000031/ $^{\circ}\text{F}$ (0.0000056/ $^{\circ}\text{C}$), while the all quartz aggregates gave the highest coefficient, 0.0000058/ $^{\circ}\text{F}$ (0.0000104/ $^{\circ}\text{C}$). Intermediate values were found for various combinations of limestone with quartz aggregates. Temperatures varied from 15 to 70 $^{\circ}\text{F}$ (-9 to 21 $^{\circ}\text{C}$).

Saemann and Washa (7) conducted tests using mortar and concretes with temperatures varying from -70 to 450 $^{\circ}\text{F}$ (-57 to 232 $^{\circ}\text{C}$). The specimens were moist-cured at normal temperatures for 14 days, stored in air at 70 $^{\circ}\text{F}$ (21 $^{\circ}\text{C}$) and 50 per cent relative humidity for 13 days and then held at test temperatures for 24 hours before tested. The coefficients varied; for the mortars from 0.0000053 to 0.0000058/ $^{\circ}\text{F}$ (0.0000095 to 0.0000104/ $^{\circ}\text{C}$) and for the concretes from 0.0000039 to 0.0000044/ $^{\circ}\text{F}$ (0.000007 to 0.0000079/ $^{\circ}\text{C}$), with the water-cement ratio varying from 0.84 to 0.48.

The influence of the amount of aggregate in the concrete was investigated by Tokuda and Ito (8). The coefficients for concretes with water-cement ratios of 0.42 varied from 0.0000045 to 0.0000066/ $^{\circ}\text{F}$ (0.0000085 to 0.0000120/ $^{\circ}\text{C}$) for aggregate contents from 0.8 to 0.4 by absolute volume.

DISCUSSION AND CONCLUSIONS

The applicability of the results presented are to concretes made with normal Portland cement, with a maximum aggregate size of 3/4 in (19 mm), with aggregate gradings that meet ASTM C 33-71 (19) and with similar petrographic analyses to that of this investigation.

Average thermal coefficients of expansion were reported by Mitchell (6): 0.0000034/ $^{\circ}\text{F}$ (0.0000061/ $^{\circ}\text{C}$) for limestone, 0.0000038/ $^{\circ}\text{F}$ (0.0000068/ $^{\circ}\text{C}$) for feldspar, and 0.000007/ $^{\circ}\text{F}$ (0.0000126/ $^{\circ}\text{C}$) for quartz. Mortars having the same water-cement ratio and aggregate content gave very similar thermal coefficients; for limestone 0.0000046/ $^{\circ}\text{F}$ (0.0000083/ $^{\circ}\text{C}$) and for feldspar 0.0000042/ $^{\circ}\text{F}$ (0.0000076/ $^{\circ}\text{C}$) (6). It can

be concluded that limestone and feldspar can be considered to have equivalent thermal properties.

The aggregate content of the concrete mixes used in this investigation varied from 69 to 72 per cent by weight. The limestone-feldspar component formed 71 to 74 per cent of the total aggregate and with quartz about 20 per cent of the total aggregate. Normal combinations of quartz sands with crushed limestone in conventional concrete mixes indicate aggregate contents from 76 to 85 per cent by weight, with about 70 per cent limestone and the remainder quartz and other minerals. It can be concluded that the results reported here would be representative of conventional concrete mixes using crushed limestone and normal quartz sands.

At normal laboratory temperatures, the dynamic modulus is generally greater than the static modulus of elasticity of concrete. At 7 days of age; variation of E_{cd}/E_c is from 1.13 to 1.25, as shown in Figure 8. The behaviour at 28 days is very similar for both wet- and dry-cured concrete, with the relation $E_{cd}/E_c = 0.95 + 0.333 (W/C)$ being indicated over the range 1.10 to 1.20. At 84 days of age; the wet-cured concrete showed a non-linear trend increasing sharply to $W/C = 0.5$ where it levelled off at about $E_{cd}/E_c = 1.10$ for greater W/C ratios. The dry-cured concrete at this age showed random error tendencies with a mean $E_{cd}/E_c = 0.98 \pm 0.02$. The use of the dynamic modulus of concrete would generally overestimate stresses computed from thermal deformations or strains, and therefore would be conservative.

The dynamic modulus increases with decrease in temperature (Figure 9). The relation is linear from 150°F (66°C) to the freezing point of water in concrete, where there is a sharp increase in the modulus produced by the frozen water. The behaviour is linear from the freezing point down to -100°F (-73°C). These slopes, dE_{cd}/dT , for the above and below freezing temperatures have been summarized in Figures 10 and 11, for dry- and wet-cured concrete respectively. For dry-cured concrete; the above freezing slopes decrease about 30 per cent while there is about a 65 per cent decrease in the below freezing slopes, from 7 to 84 days of age. For wet-cured concrete; the above freezing slopes increase about 20 per cent while there is about a 10 per cent decrease in the below freezing slopes, from 7 to 84 days of age. The slopes, above and below freezing, decrease almost linearly with a decrease in the water-cement ratio.

The average freezing point was 25°F (-4°C), with the range from 32 to 17°F (0 to -8°C). The upper limit being reached at early ages and with the greater water-cement ratios.

Concrete has a linear deformation-temperature relationship for both above and below freezing temperatures. The above freezing slope or thermal coefficient of expansion is greater than the below freezing coefficient. The coefficients increase with age. For dry-cured concrete; the above freezing coefficients increase about 12 per cent with the below freezing coefficients increasing about 20 per cent, from 7 to 84 days of age. Similar values for the wet-cured concrete are 7 and 13 per cent respectively. The dry-cured concrete has the greater increase

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in the coefficient with age. At 84 days; the coefficients of dry-cured specimens are on the average 6 per cent greater than those for comparable wet-cured specimens. The coefficients increase with a decrease in the water-cement ratio. The average increase in the coefficients for $W/C = 0.67$ to 0.45 , is 30 per cent for the above freezing coefficients and 20 per cent for the below freezing coefficients, for both dry- and wet-cured concrete.

It would seem that for concretes that are very similar to those used in this investigation, where limestone predominates in the coarse aggregate and quartz in the sand, the results could be applied directly with perhaps only small changes. For concretes that are substantially different in mineral content and/or mix proportions these results would only be approximations. These results would lie intermediate between the higher values for concretes having all quartz aggregate and those with all limestone or equivalent aggregate types. It would be recommended that for these concretes the coefficient of expansion and the modulus of the concrete should be determined experimentally to more closely approximate their deformation-temperature behaviour.

ACKNOWLEDGEMENT

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REFERENCES

1. "Manual of Steel Construction," 7th Edition, American Institute of Steel Construction, Inc., New York, N.Y., 1970, pp.6-10 to 6-12.
2. Day, E.E. and Sevand, A.H. "Characteristics of Electric Strain Gages at Low Temperatures," Proceedings, Society for Experimental Stress Analysis, V. 8, No. 1, pp.133-142.
3. Majors, Jr. H. "Characteristics of Wire Gages under Various Conditions," Proceedings, Society for Experimental Stress Analysis, V. 9, No. 1, pp.123-140.
4. Verbeck, G.J. and Hass, W.E. "Dilatometer Method for Determination of Thermal Coefficient of Expansion of Fine and Coarse Aggregate," Proceedings, Highway Research Board, V. 30, Sept. 1951, pp.187-193.
5. Callan, E.J. "Thermal Expansion of Aggregates and Concrete Durability," ACI Journal, Proceedings, V. 48, No. 6, Feb. 1952, pp.485-504.
6. Mitchell, L.J. "Thermal Expansion Tests on Aggregates, Neat Cement, and Concretes," Proceedings, ASTM, V. 53, 1953, pp.963-977.
7. Saemann, J.C. and Washa, G.W. "Variation of Mortar and Concrete Properties with Temperature," ACI Journal, Proceedings, V. 54, No. 20, November 1957, pp.385-395.

8. Tokuda, H. and Ito, T. "Some Experimental Results on Thermal Properties of Concrete," Japan Congress on Testing Materials, 7th Proceedings, 1964, pp.122-123.
9. Monfore, G.E. and Lentz, A.E. "Physical Properties of Concrete at Very Low Temperature," Journal of the PCA Research and Development Laboratories, V. 4, No. 2, May 1962, pp.33-39.
10. Cran, J.A. "An Investigation of the Low Temperature Properties of Concrete and Reinforced Concrete Slabs," M.Sc. Thesis, University of Manitoba, Winnipeg, 1963.
11. Berwanger, C. "The Modulus of Concrete and Coefficient of Expansion of Concrete and Reinforced Concrete at Below Normal Temperatures," ACI special publ., Temperature and Concrete, No. SP 25-8, 1971, pp.192-233.
12. Walker, S., Bloem, D.L. and Mullen, W.G. "Effects of Temperature Changes on Concrete as Influenced by Aggregates," ACI Journal, Proceedings, V. 48, No. 8, Apr. 1952, pp.661-678.
13. Dettling, H. "Die Warmedehnung des Zemensteines, der Gesteine und der Betone," Deutscher Ausschuss fuer Stahlbeton, V. 164, 1964, pp.1-64.
14. Berdichevskii, A.I. and Svirdov, N.V. "Prochnost Izzibaumykh Zhelezobetonnykh Elementov..." "Flexural Strength of Reinforced Concrete Members at very Low Temperatures), Beton i Zhelezobeton, V. 1, January 1965, pp.16-21.
15. Moskin, V.M., Kapkin, M.M. and Anatonov, L.N., "Vliyanie Otriatelnykh Temperatur na Prochnost i Uprugoplasticheskii Svoista Betona" (Effect of Freezing Temperature on Strength and Elastoplastic Properties of Concrete), Beton i Zhelezobeton, V. 10, October 1967, pp.18-21.
16. Cruz, R.C. "Elastic Properties of Concrete at High Temperatures," Journal of the PCA Research and Development Laboratories, V. 8, No. 11, January 1966, pp.37-45.
17. Hansen, C.T. and Erikson, L. "Temperature Change Effect on Behaviour of Cement Paste, Mortar and Concrete Under Load," ACI Journal, Proceedings, V. 63, No. 4, April 1966, pp.489-504.
18. "Standard Specification for Portland Cement," ASTM C 150-71, ASTM Standards, Part 10, 1972, pp.99-101.
19. "Standard Specification for Concrete Aggregates," ASTM C 33-71a, ASTM Standards, Part 10, 1972, pp.14-22.
20. "Standard Method of Test for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens," ASTM C215-70, ASTM Standards, Part 10, 1972, pp.134-137.

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21. "Standard Method of Test for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression," ASTM C469-70, ASTM Standards, Part 10, 1972, pp.277-281.
22. Philleo, R.E. "Comparison of Results of Three Methods for Determining Young's Modulus of Elasticity of Concrete," ACI Journal, Proceedings, V. 51, No. 5, Jan. 1955, pp.461-472.
23. Verbeck, G. and Klieger, P. "Calorimeter-Strain Apparatus for Study of Freezing and Thawing Concrete," Bulletin No. 176, Highway Research Board (Washington), 1958, pp.9-22.
24. Sarkar, A.F. "The Behaviour of Concrete and Reinforced Concrete Under Short Term Steady State Temperatures," M.A.Sc. Thesis, University of Ottawa, Ottawa, Canada, 1972.

APPENDIX-NOTATION

- dE_{cd}/dT = slope of dynamic modulus versus temperature
- E_c, E_s = static modulus of elasticity of concrete and steel respectively
- E_{cd} = dynamic modulus of elasticity of concrete
- f'_c = concrete strength, 6 x 12 in. (15.2 x 30.5-cm.) cylinders
- K_c, K_s = coefficient of expansion of concrete and steel respectively
- L = length of prism for SCT5 test
- n = resonant or natural frequency of longitudinal vibration of concrete prism, cps
- RH = relative humidity
- T = temperature
- W = weight of the prism
- W/C = water-cement ratio



Fig. 1 - Environmental Chamber: Prisms seen through windows, with thermocouple potentiometer at right, and optical bench

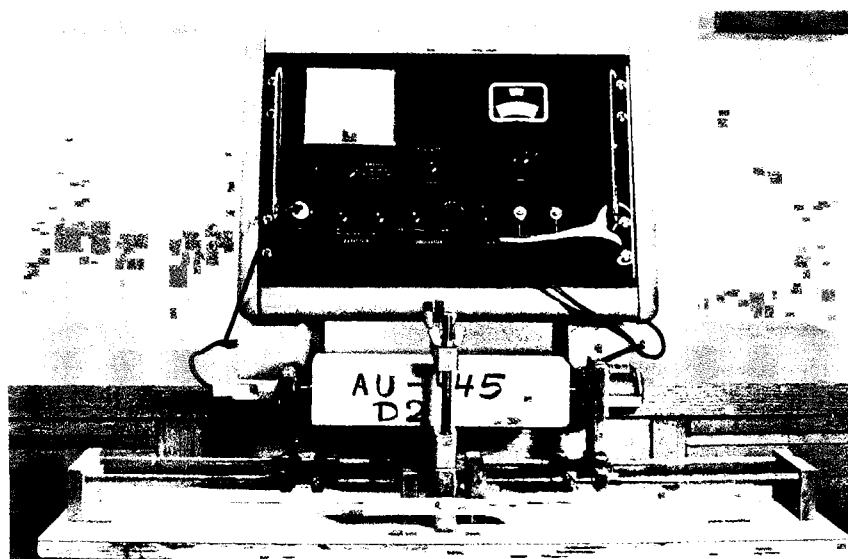


Fig. 2 - Electrodynamics Materials Tester: Vibrator and pick-up touch ends of prism clamped in the bench and are connected to the SCT 5

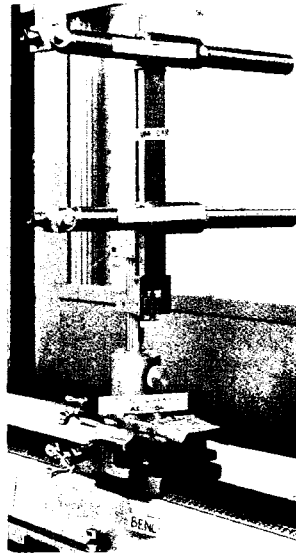


Fig. 3 - Optical Extensometer: Cathetometers bar mounted on carriage attached to invar which moves on the optical bench

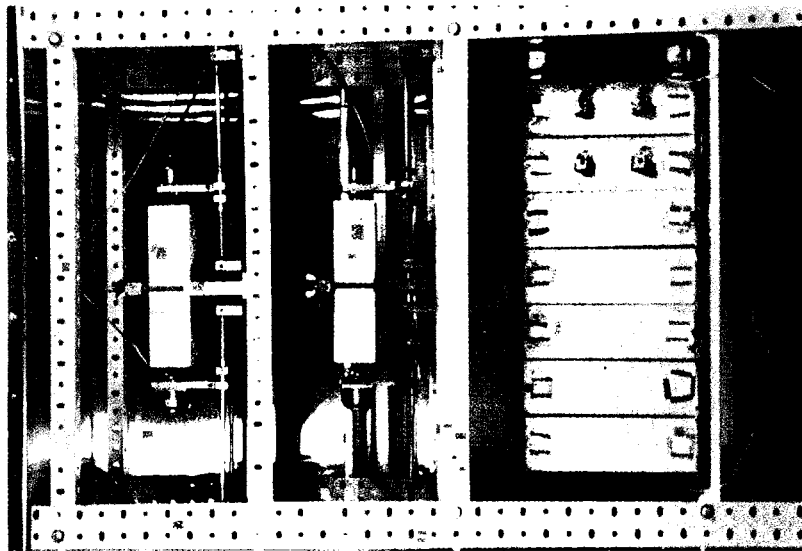


Fig. 4 - Internal View of Chamber: Prisms mounted in SCT 5 test benches on top two shelves, with coefficient of expansion prisms on lower platform



Fig. 5 - Aluminum Control Plate: Targets for cathetometers at top and bottom, with DCDT mounted in the centre

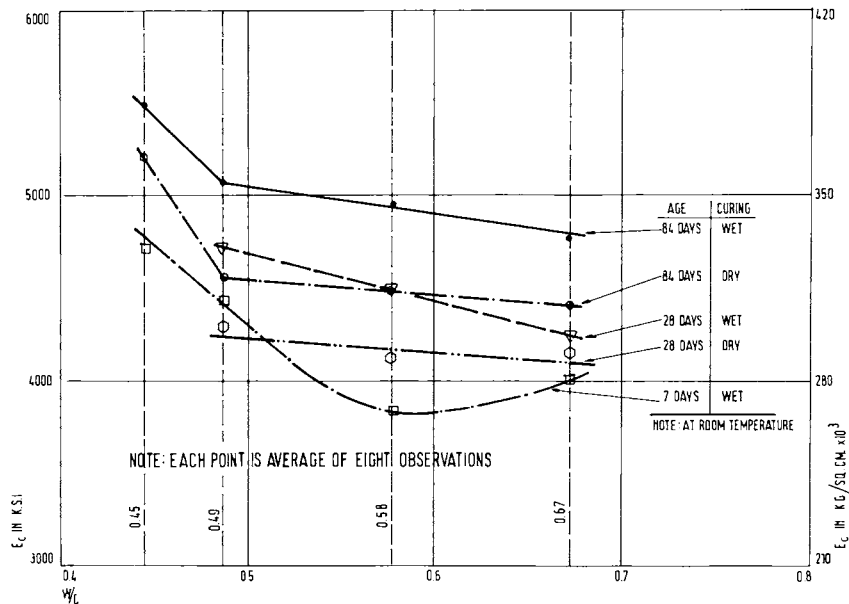


Fig. 6 - Effect of Water-Cement Ratio on Static Modulus of Elasticity of Concrete

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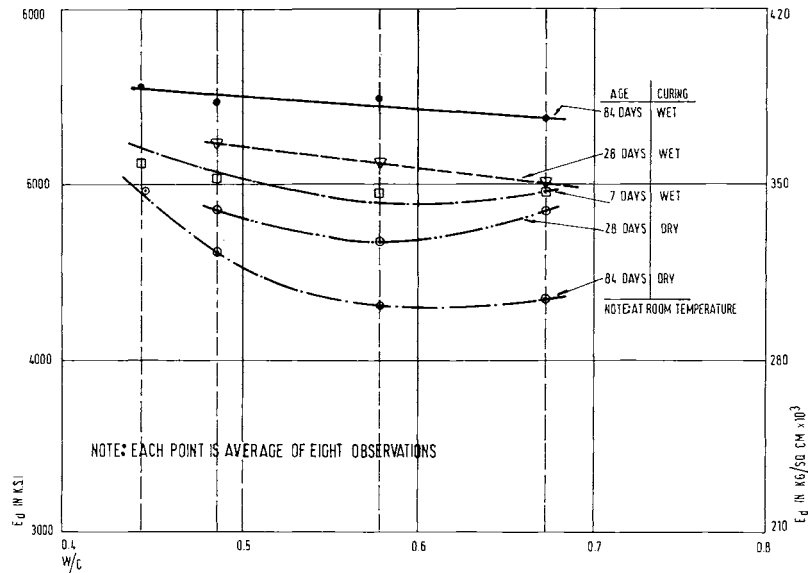


Fig. 7 - Effect of Water-Cement Ratio on Dynamic Modulus of Elasticity of Concrete

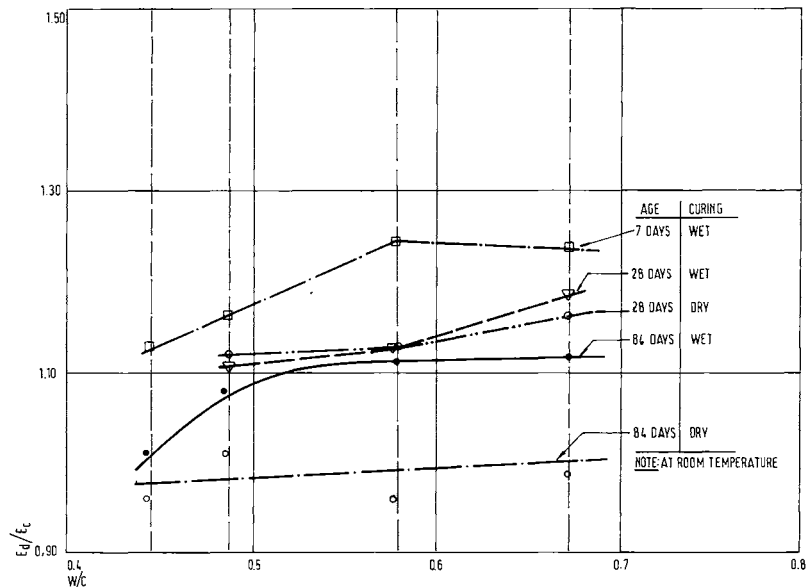


Fig. 8 - Variation of Ratio of Dynamic to Static Modulus of Elasticity of Concrete with Water-Cement Ratio

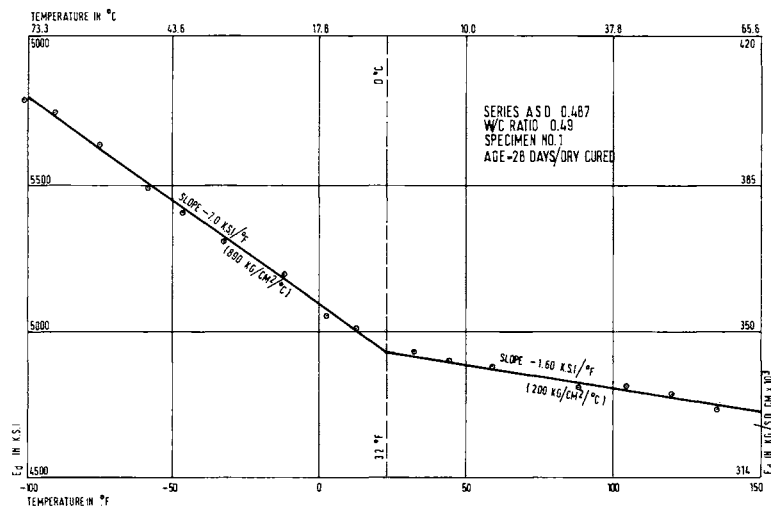


Fig. 9 - Effect of Temperature on Dynamic Modulus of Elasticity

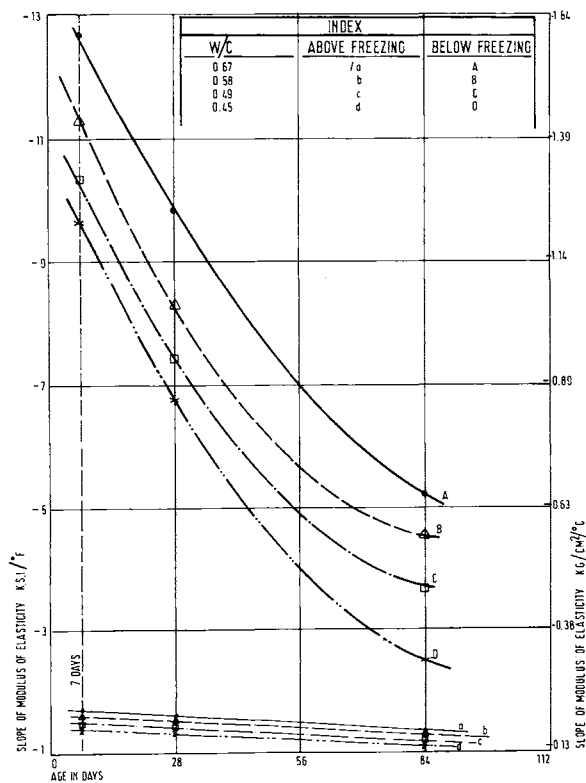


Fig. 10 - Effect of Age on Slope of Dynamic Modulus of Elasticity Versus Temperature Relation for Dry-Cured Concrete

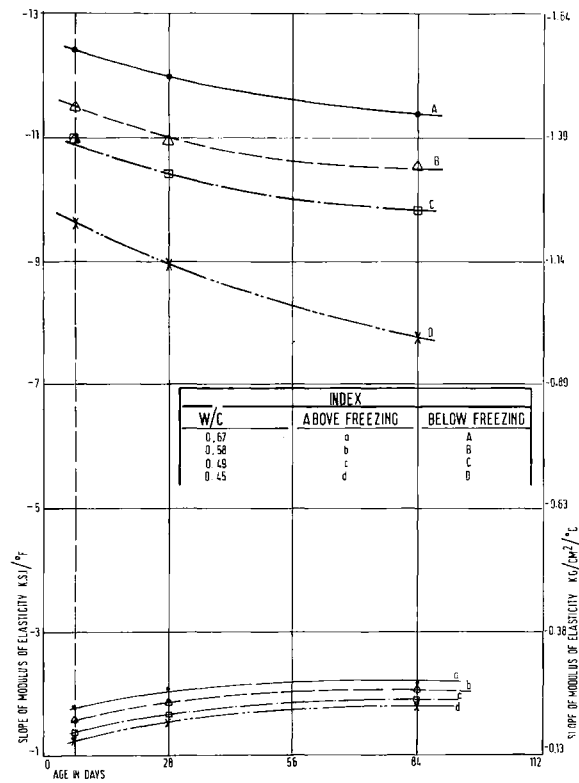


Fig. 11 - Effect of Age on Slope of Dynamic Modulus of Elasticity Versus Temperature Relation for Wet-Cured Concrete

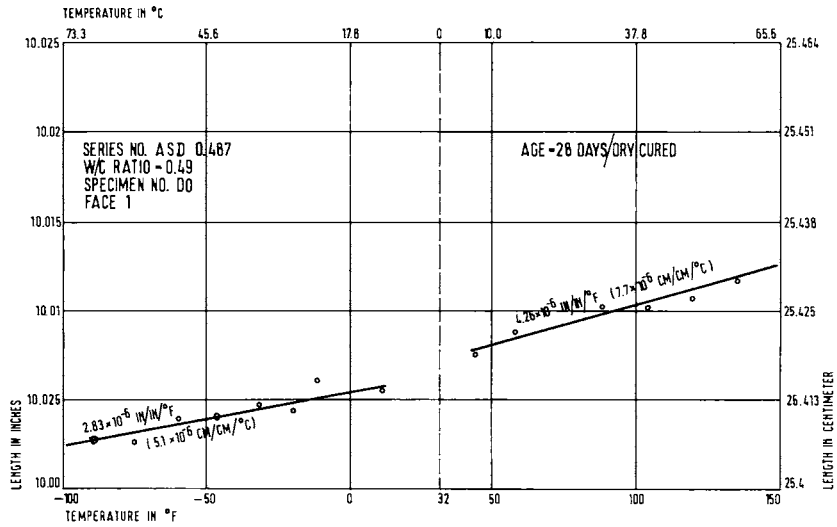


Fig. 12 - Effect of Temperature on the Coefficient of Thermal Expansion of Concrete

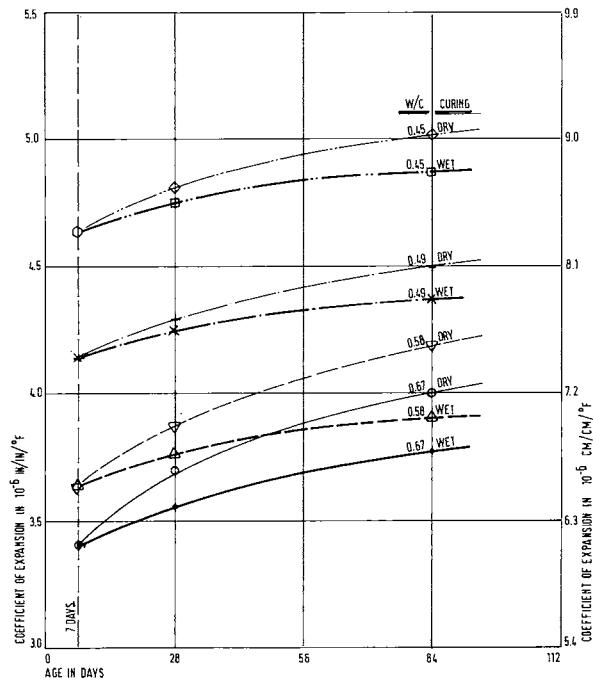


Fig. 13 - Effect of Age on the Coefficient of Thermal Expansion of Concrete for Above Freezing Temperatures

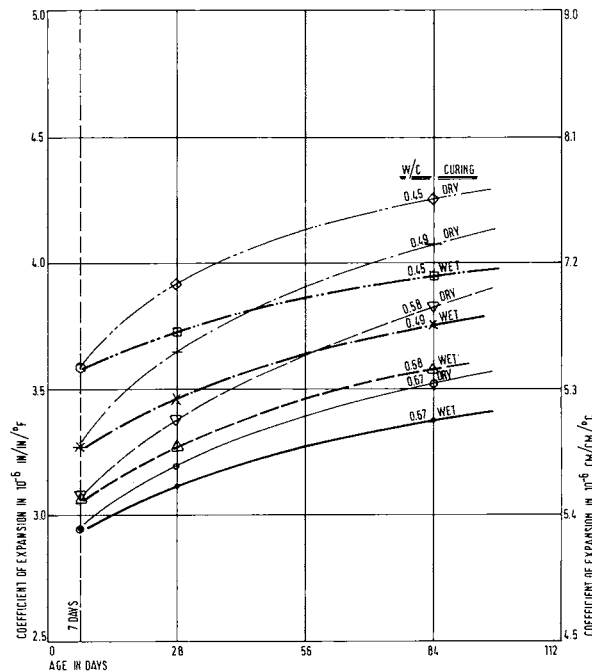


Fig. 14 - Effect of Age on the Coefficient of Thermal Expansion of Concrete for Below Freezing Temperatures

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EARLY FREEZING OF VACUUM PROCESSED CONCRETE

BY
BRIAN B. HOPE
PETER J. QUELCH

Synopsis: The effect of freezing temperatures on the compressive strength and ultrasonic pulse velocity of vacuum processed and non-processed concrete was investigated for two concretes with initial water/cement ratios of 0.71 and 0.45. The average reduction in water/cement ratio of 6 in. x 12 in. (15.2 x 30.5 cm.) cylinders obtained by vacuum processing was 0.18 and 0.09 respectively. Vacuum processed and non-processed cylinders were stored in air maintained at 0°F (-18°C) after initial curing periods of 0, 3, 6, 12 or 24 hours at 70°F (21°C) and 100% relative humidity. The resistance to damage by freezing of both concretes was enhanced by vacuum processing. For both mixes the required curing time so that there was no significant loss in the 28-day strength of 6 in. x 12 in. (15.2 x 30.5 cm.) cylinders was halved by vacuum processing.

Keywords: compressive strength; concretes; freezing; ultrasonic tests; vacuum treated concrete.

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ACI member Brian B. Hope is associate professor of civil engineering, Queen's University at Kingston, Ontario. He is secretary of ACI Committee 548, Polymers in Concrete and a member of Committee 209, Creep and Shrinkage in Concrete.

Peter J. Quelch is assistant engineer, Canadian National Railways, Montreal, Quebec. He holds M.Sc. and M.B.A. degrees from Queen's University at Kingston, Ontario.

INTRODUCTION

The problems of concreting in winter have been approached largely in terms of methods to improve curing conditions. Techniques such as steam heating have been developed and used successfully but are expensive to install. The use of vacuum processing was investigated and is reported here as a possible technique for reducing the curing time required for frost protection of concrete and, thereby, reducing the need for long periods of artificial temperature control.

Vacuum processing was developed and patented, according to Lockhardt (1), by K.P. Billner. It is essentially a method for removing excess water from fresh concrete by the application of a partial vacuum. The technique has the advantages of being relatively inexpensive to install and of forming concrete which gains strength at a very rapid rate. Vacuum processing permits the use of light formwork (2) which can often be stripped immediately after processing and produces a concrete with high resistance to abrasion (3).

The objective of the test program described here was to investigate the advantage of vacuum processing on concrete which was to be subjected to early freezing. For this investigation two different concretes were designed having nominal 28-day strengths of approximately 3000 psi (210 kg/cm²) and 5000 psi (350 kg/cm²). For each of these concretes seven batches were made; two were used for control purposes and the remaining five placed in a freezing chamber 0, 3, 6, 12 or 24 hours after mixing and kept there for three days. The criteria used to investigate the deterioration of the concrete were the pulse velocity and the compressive strength.

TESTING PROCEDURE

Materials

The concrete was made from an ordinary Portland Cement (Type 1), siliceous natural sand with a fineness modulus of 2.63 and river washed granite of 3/8 in. (9.5 mm) maximum size.

Preparation of Specimens

Two mixes were designed, mix 1 with a water/cement ratio of 0.71, and mix 2 with a 0.45 water/cement ratio. Mix proportions are shown in Table 1. Mixing and molding procedures were standardized throughout the test series. The mean values of slump, air content and wet density for mix 1 were 2.2 in. (5.6 cm), 3.7 percent and 143.2 p.c.f. (2295 kg/m³) respectively. The mean values for mix 2 were 3.2 in. (8.1 cm) slump, 3.5 percent air content and 146.6 p.c.f. (2350 kg/m³) wet density.

Seven batches of each mix were made and from each batch twelve 6 in. diameter x 12 in. long (15.2 x 30.5 cm), and thirteen 4 in. x 8 in. (10.2 x 20.3 cm) cylinders were cast. Six of the 6 in. (15.2 cm) diameter specimens from each batch were vacuum processed. The remaining cylinders were not vacuum processed and, whether they are frozen or not, are referred to as non-processed cylinders.

Vacuum Processing

Two of the cylinder molds used for vacuum processing are shown in Figure 1. Wire mesh was welded to the inside surface of each cylinder and a rubber gasket placed on the base plate. To prepare the cylinder for use the wing nuts on the vertical flange were made finger tight and a filter cloth of unbleached linen was stretched over the wire mesh and held in place with paper clips, as shown in Figure 2. After the vacuum hoses were connected each cylinder was filled with concrete and the vacuum pump started. As a partial vacuum developed in the small chamber formed by the wire mesh a strip of sponge rubber between the vertical flanges was compressed and the concrete slumped somewhat. One half minute after processing was commenced the wing nuts on the vertical cylinder flange were retightened and the cylinder was topped-up with concrete. This arrangement of the vacuum cylinder permitted a limited degree of radial compaction (as the sponge rubber compressed) together with axial compaction caused by atmospheric pressure on the open end of the cylinder.

In order that three cylinders could be processed from each batch, at more or less the same time, each specimen was vacuum processed for ten minutes in three stages of duration 3 minutes, 3 minutes and 4 minutes respectively. The time between stages was from six to eight minutes. The amount of water withdrawn was measured for each individual cylinder by determining the amount of water retained by separators placed in the vacuum line between the cylinder and the pump for this purpose. On completion of vacuum processing the specimens were removed from their molds and wrapped in plastic sheeting to prevent loss of moisture.

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Curing Conditions

a. Control specimens -- Two of the seven batches produced for each mix were used for control purposes. Specimens of the control batches were stored in a standard curing room maintained at 70°F (21°C) and 100% relative humidity until age 3 days. Between the ages of 3 and 6 days the concrete cylinders were stored in water at a constant temperature of 70°F (21°C). On removal from the water tank specimens were returned to the curing room until tested in compression.

b. Frozen specimens -- The remaining five batches of each of mixes 1 and 2 were stored in the curing room for 0, 3, 6, 12 or 24 hours after molding. On completion of this initial curing phase the cylinders were transferred to a freezing chamber maintained at 0°F (-18°C) for 3 days. Hydration of cement paste is almost completely halted when concrete is stored at 0°F (-18°C) (4,5) and consequently the 3-day freezing period is omitted from the age calculations of the concrete.

After completion of the cold storage phase specimens were transferred to the curing room where they remained until they reached an age of 3 days. Then, as for the unfrozen cylinders, the specimens were soaked for 3 days in water and returned to the curing room until tested in compression. The 3-day water storage procedure was adopted in order to reduce the difference in degree of saturation between frozen and non-frozen specimens. This storage procedure was not effective, of course, for the tests at 3 days age and it is believed that differing degrees of saturation affected the 3-day results as discussed later.

Figure 3 is a typical time-temperature diagram which illustrates the curing procedure for specimens initially cured for 12 hours.

Specimen Usage

a. Control mixes -- The 3, 7 and 28-day strengths for the 6 in. (15.2 cm) diameter non-processed unfrozen control mixes are the mean values obtained from four specimens.

In addition to the specimens tested above, four 4 in. (10.2 cm) diameter specimens were used to check the batch to batch variations.

The control pulse velocity readings at 3, 7 and 28 days are respectively the mean values of twelve, eight and four 'specimen mean values'. A pulse velocity 'specimen mean value' is the mean of three pulse velocity readings taken on a single specimen as described below.

b. Frozen mixes -- The compressive strength of frozen cylinders is based on the mean value obtained from three specimens.

The pulse velocity of frozen specimen at 3, 7 and 28 days are respectively the means of six, three and three 'specimen mean values'.

Testing Procedure

The compressive strength was determined by loading specimens of 6 in. and 4 in. (15.3 cm and 10.2 cm) diameter at rates of 0.05 in. per minute and 0.025 in. per minute (1.25 and 0.62 mm per minute) respectively.

The ultrasonic pulse travel time was measured along three transverse paths for each specimen. The centres of the path locations were, 1 in. (2.54 cm) from the top and bottom of the specimen and at the mid height. The mean of these three results was the 'specimen mean value'.

Method of Comparison of Frozen and Control Specimens

The basis of all comparisons described here is the mean value of compressive strength and pulse velocity.

The uniformity of concrete batches was studied using four control cured 4 in. (10.2 cm) diameter cylinders. In all cases except two the probability that all specimens came from the same population was statistically significant using 95% confidence limit. The two batches that fell outside the 95% confidence limit were the batches that were initially cured prior to freezing for 6 hours and 12 hours from mix 2.

RESULTS AND DISCUSSION

The Effects of Vacuum Processing

a. The control specimens -- The average reduction in water/cement ratio obtained through vacuum processing was 0.18 or 25%, for mix 1 and 0.09, or 20%, for mix 2.

The amounts of water withdrawn from the concrete decreased significantly as processing progressed. On the average, of the water collected in the primary separator only 8% and 3% was collected for mixes 1 and 2 respectively in the last four minutes of processing. The total water removed from mix 1 specimens was about 50% higher than that removed from mix 2 specimens.

The compressive strength of all 6 in. (15.3 cm) diameter control specimens is shown in Figure 4. The vertical lines A-A and B-B mark the initial water/cement ratios of mixes 1 and 2 respectively. Results between A-A and B-B are for vacuum processed specimens from mix 1 and those to the left of B-B are for vacuum processed specimens from mix 2. The curves drawn connect points of equal age and were drawn through the results for both processed and non-processed specimens. This was done because there

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was no reason to suspect that the compressive strength versus water/cement ratio functions were different for processed and non-processed concretes.

For mix 1, the 3-day old vacuum processed cylinders had average compressive strengths 66% higher than non-processed cylinders. Mix 2 3-day old vacuum processed specimens had strengths 41% higher than non-processed specimens. The high early strength potential of vacuum processed concrete is also demonstrated by the fact that, at 3 days, the average vacuum processed cylinder of mix 2 had a 4% higher strength than a similar non-processed cylinder at 28 days. The mean 3-day strength of mix 1 vacuum processed concrete was 60% of the 28-day non-processed strength.

In contrast to the large increases in strength, the difference in the pulse velocity measurement of processed and non-processed specimens is very small. The processed cylinders had pulse velocities 2% to 3% higher for mix 1 and 1% to 5% higher for mix 2 compared with the non-processed specimens.

The small difference in pulse velocity between processed and non-processed specimens can be explained on the basis of the constituent composition of the cylinders. Even though some water is withdrawn from the processed cylinders the remaining constituents were present in the same proportion. The density of the processed concrete was not significantly different from that of the non-processed concrete as a result of incomplete compaction of the specimens as water was removed during vacuum processing. As water is removed the concrete becomes less fluid and compaction ceases when atmospheric pressure is insufficient to overcome internal friction. Additional water is then removed as it is expelled by the expansion of entrapped air as the pressure in the specimen drops. The density remains unchanged if the concrete is compacted an amount equal to 43 percent of the volume of water removed. For mixes 1 and 2, which contained about 3.5 percent entrapped air, this could be accommodated by reductions in the entrapped air pressure of 15 in. and 6 in. (38 cm and 15 cm) of mercury respectively. Thus although water was removed during processing, provided sufficient remained for hydration the concrete matrices would be initially similar so far as the pulse velocity test is concerned.

b. The frozen specimens -- The average compressive strength of frozen specimens versus initial curing time (curing time prior to freezing) is shown in Figures 5 and 6 for mixes 1 and 2 respectively. The strength of the frozen specimens is expressed as a percentage of the 28-day strength of non-frozen specimens. In both figures the heavy horizontal lines indicate the non-frozen 3 and 28-day non-processed compressive strength, as determined from the concrete mixes.

The results for mix 1 indicate that a higher degree of protection from freezing temperatures is obtained by vacuum processing. In terms of absolute strength at 3 days, processed

concrete with 3 hours initial curing had a higher compressive strength than 3-day old plain concrete which has not been frozen.

The compressive strength results for mix 2 shown in Figure 6 are more difficult to interpret. Here the vacuum processed concrete shows an apparently greater strength at 3 days age than at 28 days age for specimens initially cured 3, 6 and 12 hours. The reason for this anomalous behavior is likely to be the effect of the degree of saturation of the specimens at the time of testing. Saturated specimens show a considerably lower strength when tested in compression compared with similar specimens which are not saturated.

As far as mix 2 is concerned specimens from this mix which were vacuum processed attained strengths of 4980 psi (349 kg/cm^2) at 3 days and 5420 psi (381 kg/cm^2) at 28 days. The high 3-day strength is indicative of a well compacted concrete with a low water/cement ratio which does not contain a system of interconnected capillary pores. Indeed, with a water/cement ratio of 0.36, which was attained for this mix by vacuum processing the capillary pore system becomes discontinuous at a maturity factor of about 0.38 (6) which occurs after about 20 hours hydration.

The vacuum processed cylinders were not saturated at the end of the processing operation as indicated by the density measurements. In addition moisture may have been lost during the period the specimens were frozen. With a discontinuous capillary pore system saturation is not obtained quickly upon exposure to moisture in a curing room. Consequently it is believed that the frozen specimens tested at age 3 days were not saturated at the time of testing. This resulted in an indicated strength higher than that which would have been obtained had the specimens been saturated.

The above explanation, based on degree of saturation, of the occurrence of an apparently higher 3-day strength than 28-day strength is reinforced by consideration of the pulse velocities shown in Figures 7 and 8. The pulse velocity, expressed as a percentage of the pulse velocity of 28-day non-frozen specimens, is plotted against initial curing time for mixes 1 and 2 in these figures. The pulse velocity of vacuum processed concrete is always higher than the pulse velocity of non-processed concrete, the difference being particularly significant at 3 days age. It is also interesting to note that for mix 2, shown in Figure 8, the vacuum processed concrete had a higher pulse velocity at 28 days than at 3 days for all initial curing times. Therefore, for the vacuum processed concrete of mix 2 with 3, 6 and 12 hours curing before freezing the 3-day pulse velocity is lower than that at 28 days even though compression tests indicated that the strength was apparently higher at 3 days than 28 days. This result is entirely in keeping with the hypothesis that these specimens were not saturated at 3 days age since a reduction in moisture content causes a significant drop in pulse velocity. Consequently if the specimens

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were not fully saturated at 3 days age, then both a higher strength and lower pulse velocity, compared with a saturated specimen, would be the expected and normal results.

The hypothesis above has been used to explain the results for mix 2. However, the hypothesis is entirely in agreement with the results for mix 1. The key variable appears to be the time at which the capillaries become discontinuous since this controls the egress of moisture during storage at freezing temperatures and the ingress of water during subsequent curing. As far as mix 1 is concerned the capillaries do not become discontinuous at any time during the testing period. The specimens are, therefore, easily capable of reaching saturation quickly during curing after the freezing period.

Vacuum Processing for Frost Protection

Table 2 gives the initial curing time required by the two mixes in order that the results of the particular test (compressive strength and pulse velocity) are not statistically different from the non-frozen values at the 95% confidence level. It can be seen that vacuum processed concrete requires about 50% less initial curing than does non-processed concrete if compressive strength is to be the criteria for damage. However, if pulse velocity is taken as the indicator for frost damage, the initial curing time for processed concrete drops to a quarter of that required for non-processed concrete. In all cases but two, the initial curing time required for no damage differs depending upon whether the criteria is compressive strength or pulse velocity. This result indicates the importance of choosing an appropriate testing criteria when investigating the integrity of concrete.

CONCLUSIONS

The principal conclusions derived from this series of tests are -

1. Intermittent vacuum processing was effective in removing water from the concrete.
2. The compressive strength of the concrete was increased significantly by vacuum processing. The pulse velocity results were improved but not to the same degree.
3. Increased initial curing generally reduced the damage caused by freezing as indicated by compressive strength tests. In cases where increased initial curing time did not reduce damage caused by freezing, the moisture content of the concrete at the time of testing was hypothesized to have played a significant role in the outcome of the compressive strength results.

4. Vacuum processed concrete with a water/cement ratio higher than that of a plain concrete can attain a higher compressive strength if both have been subjected to freezing temperatures after the same initial curing period.
5. For the concretes and specimen sizes used in this testing program Table 2 shows the initial curing time required in order that the compressive strength of frozen and non-frozen specimens is within the 95% confidence interval.
6. Vacuum processing of concrete would seem to hold promise as a method of protecting fresh concrete from the actions of frost. Additional work in this area may provide practical solutions to some of the problems of placing concrete during winter.

ACKNOWLEDGEMENTS

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REFERENCES

1. Lockhardt, W.F. "Vacuum Concrete" A.C.I. Journal, Proceedings Vol 34, Jan-Feb 1938, pp 305-320
2. Gooch, J.N. "Vacuum Concrete" Civil Engineering and Public Works Review, Vol 50 No 586 April 1955, pp 409-411
3. Mather, B. Discussion of "Vacuum Concrete" by Kurt Billig, Journal of I.C.E. Vol 30 1948, pp 420-421
4. Möller, G. "Tests of Resistance of Concrete to Early Frost Action" R.I.L.E.M. Symposium on Winter Concreting (Copenhagen, 1956)
5. Lyse, I. R.I.L.E.M. Symposium on Winter Concreting (Copenhagen, 1956)
6. Powers, T.C., Copeland, L.E. and Mann, H.M. "Capillary Continuity and Discontinuity in Cement Pastes" Journal, P.C.A. Research and Development Laboratories, 1 No 2, pp 38-48 (May 1959)

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TABLE 1
Concrete Mix Proportions

Mix	Quantities in lb per cu yd (kg/m^3)				
	Cement	Water	Granite Aggregate	Siliceous Aggregate	Effective w/c ratio
1	507 (301)	385 (228)	1453 (862)	1453 (862)	0.71
2	778 (461)	373 (221)	1472 (874)	1280 (760)	0.45

TABLE 2
Initial Curing Time (Hours) For Frost Protection

Testing Criterion Specimen Type	Strength		Pulse Velocity	
	Mix 1	Mix 2	Mix 1	Mix 2
Vacuum Processed 6" x 12" cylinder	6	12	6	3
Non-Processed 6" x 12" cylinder	12	24	24	12
Non-Processed 4" x 8" cylinder	24	24	24	12

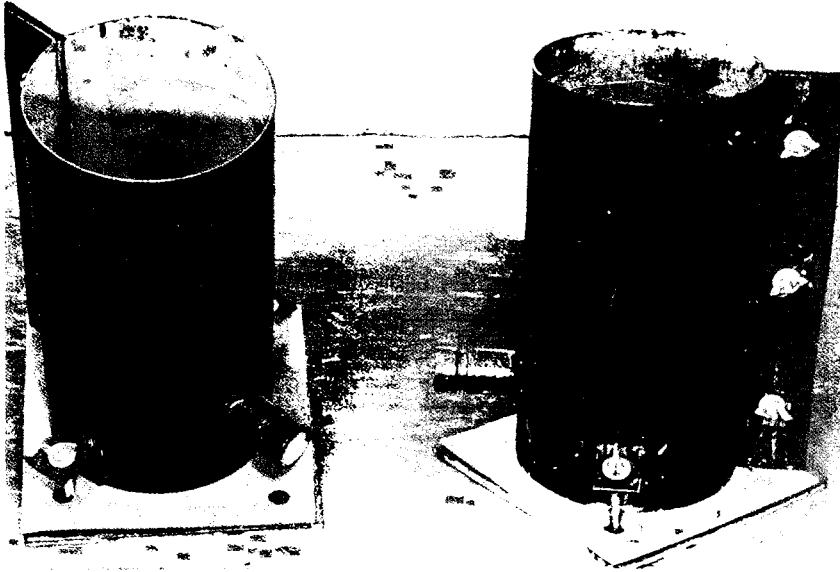


Fig.1 - Vacuum Cylinder Molds

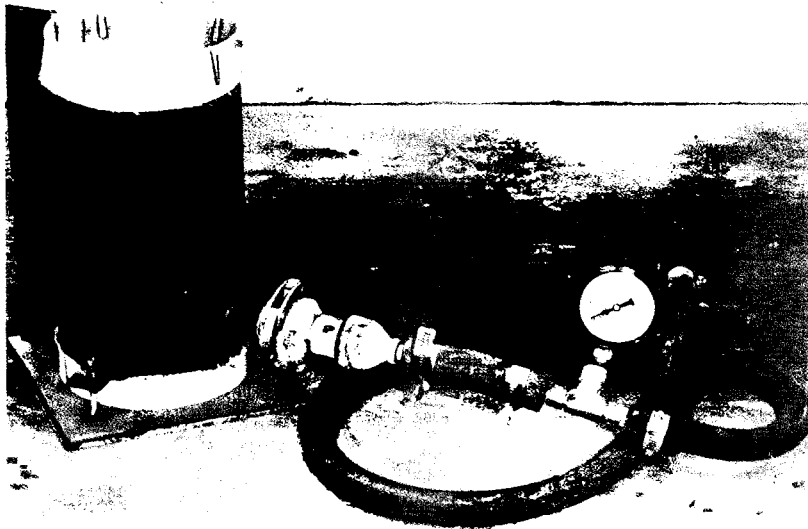


Fig.2 - Vacuum Cylinder Mold Ready for Use

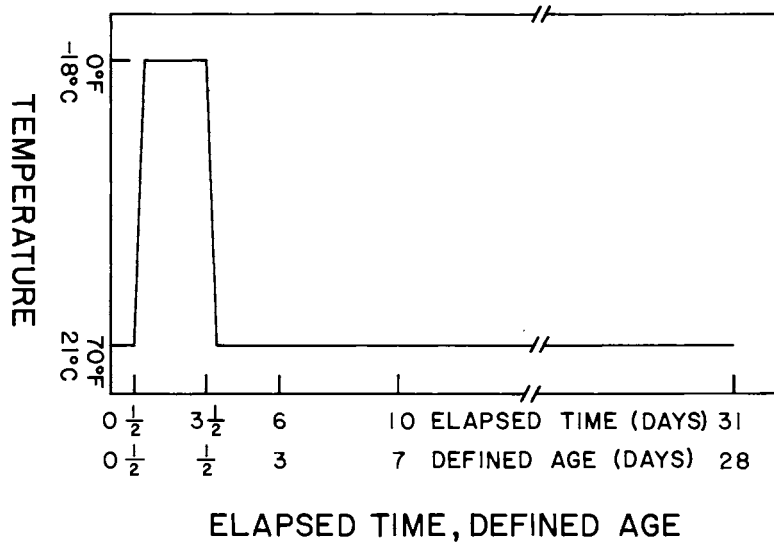


Fig. 3 — Time - Temperature diagram for concrete frozen after 12 hours initial curing

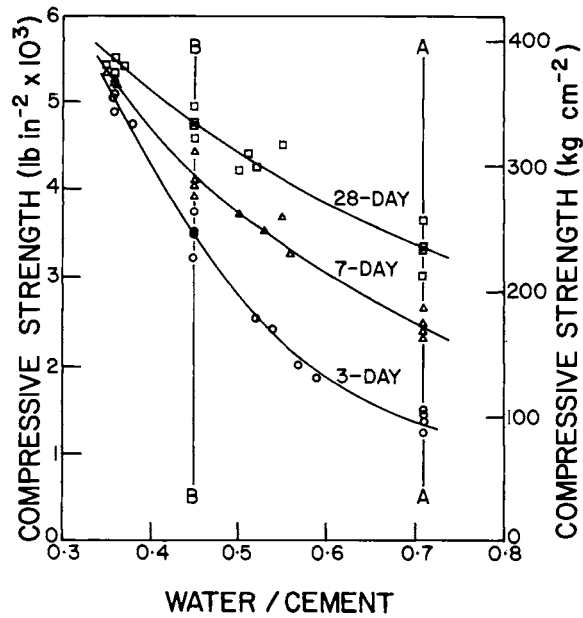


Fig. 4 — Compressive Strength of Control Specimens

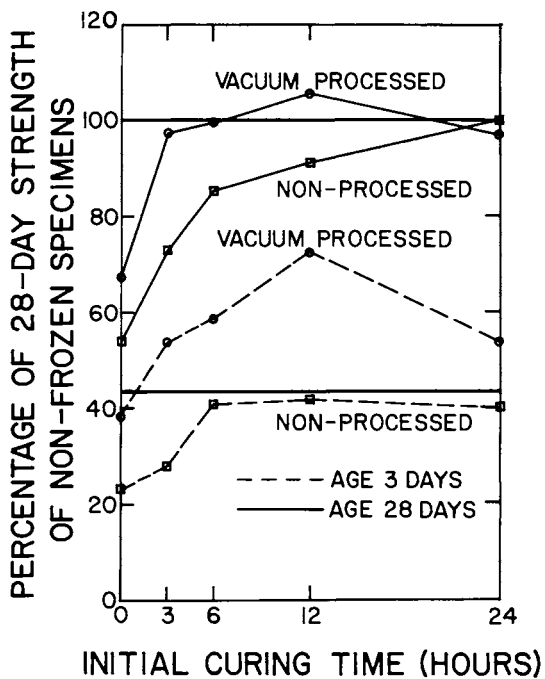


Fig.5 — Compressive Strength of Frozen Specimens from Mix 1

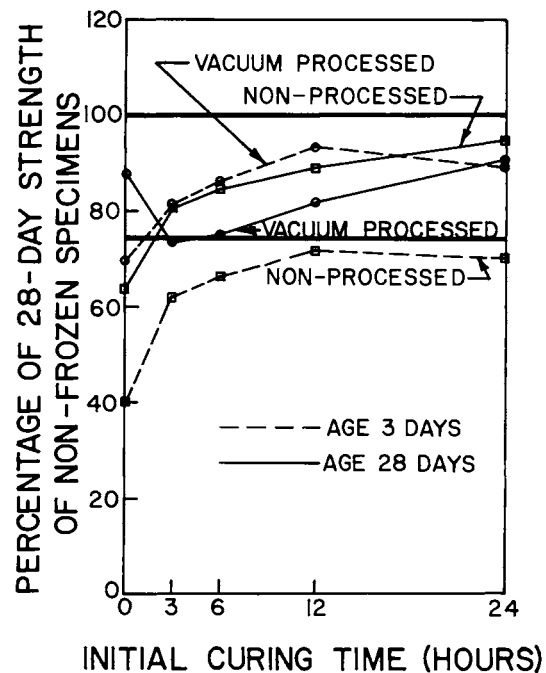


Fig.6 — Compressive Strength of Frozen Specimens from Mix 2

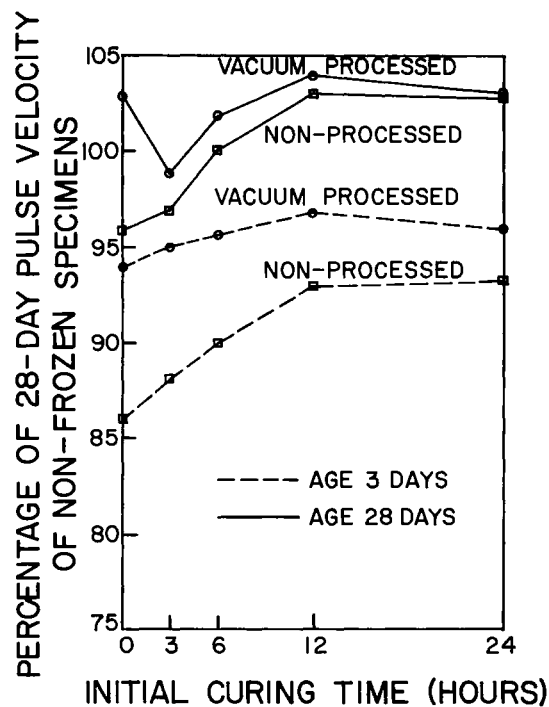
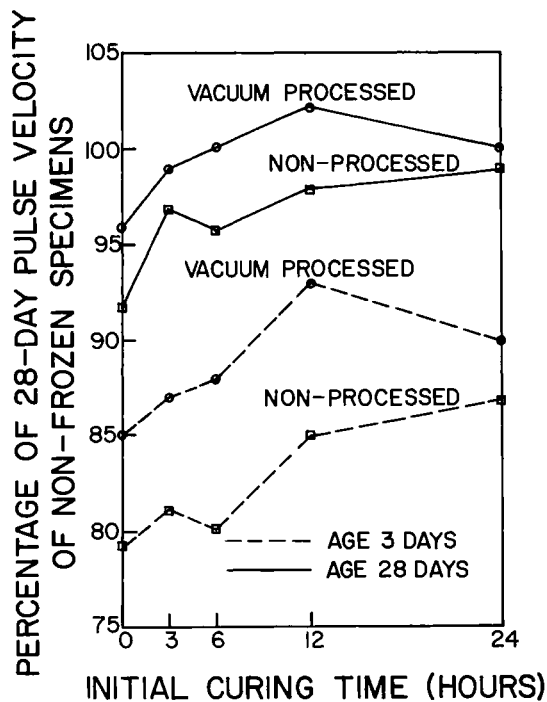


Fig. 7—Pulse Velocity of Frozen Specimens from Mix 1 Fig. 8—Pulse Velocity of Frozen Specimens from Mix 2

SP 39-3

EFFECT OF BELOW FREEZING TEMPERATURES ON STRENGTH DEVELOPMENT OF CONCRETE

BY
V. M. MALHOTRA
CARL BERWANGER

Synopsis: This paper reports the relationships of the strength of standard laboratory-cured cylinders to the strength of concrete in 24 x 24 x 66-inch (0.61 x 0.61 x 1.68-meter) columns and 24 x 24 x 8-inch (61 x 61 x 20-cm) slabs and to that of cylinders cured under exposed winter conditions. The columns, slabs and half of the cylinders were left on the outdoor exposure platform after an initial 3-day curing at 50 + 10°F (10 + 6°C). The other cylinders were cured under standard conditions. Test cores were taken from the column, prisms, and slabs after 7, 14, 28, and 91 days, and finally at about 7 months for testing in compression.

The test results show that concrete initially cured for three days at about 50°F (10°C) and 75 relative humidity and then exposed to below freezing temperatures continues to gain strength. The rate of gain in strength of field-cured concrete shows a marked increase after the ambient temperature of exposure exceeds 50°F (10°C). At the age of 7 months, the compressive strengths of the cores drilled from columns and slabs approach or exceed the compressive strengths of test cylinders cured under standard moist conditions.

Keywords: columns (supports); compression tests; compressive strength; concrete slabs; concretes; freezing; low temperature; moist curing.

V.M. Malhotra is a materials engineer, Department of Energy, Mines and Resources, Ottawa, Canada. He is past chairman of ACI Committee on Computers and is active in other technical committees of ACI.

Carl Berwanger is a professor in civil engineering, University of Ottawa, Ottawa, Canada. He is a member of several technical committees of ACI and has published over 20 technical papers.

INTRODUCTION

During the winter months, in both Canada and the northern parts of the United States, concrete is often placed when the ambient air temperature is below freezing. Concrete placed under these conditions during the construction of buildings is usually cured for about three days using plastic enclosures, inside which the temperature is maintained sufficiently high to keep the floor slab above from freezing. Following this, the heated enclosures are removed and the concrete structure is allowed to cure exposed to winter temperature extremes. Few data are available regarding the strength development of concrete in exposed structures during the winter months, when the ambient air temperatures vary between -20°F and 32°F (-29 to 0°C). The published data (1-5) indicate that measurements on laboratory-cured test cylinders neither accurately nor consistently indicate the strength development in concrete structures. The present study has been undertaken to investigate the relationship of the strength of standard laboratory-cured cylinders to the strength of concrete in structures cured under exposed winter conditions.

This investigation report is part of a co-operative research into the behaviour of concrete jointly undertaken by the Mines Branch and the Department of Civil Engineering, University of Ottawa. This project has been especially designed to determine the effect of below-freezing temperatures on the strength development of concrete.

SCOPE OF INVESTIGATION

In this study, five concrete mixes were investigated. The nominal water/cement ratio varied from 0.75 to 0.42 and corresponding cement contents varied from 360 to 700 lb/cu yd (214 to 415 kg/m³). From each mix, one 24 x 24 x 66-in. (0.61 x 0.61 x 1.68-meter) column*,

*In Mix IV and V, one 24 x 24 x 66-in. (0.61 x 0.61 x 1.68-m) horizontal prism was cast instead of the column.

one 24 x 24 x 8-in. (61 x 61 x 20-cm) slab, sixteen 6 x 21-in. (15 x 30-cm) cylinders and sixteen 4 x 8-in. (10 x 20-cm) cylinders were cast on an outdoor exposure platform. The columns, prisms, slabs, and half of the cylinders were left on the outdoor exposure platform after an initial 3-day curing at $50 \pm 10^{\circ}\text{F}$ ($10 \pm 6^{\circ}\text{C}$), achieved by the use of propane gas heaters. The other cylinders were cured under standard conditions. The ambient temperature during the casting and initial heating of the specimens, December 10-24, 1970, varied from -15°F to 25°F (-26.2 to -4.0°C). Test cores were taken from the columns, prisms, and slabs after 7, 14, 28, and 91 days, and finally at about 7 months. The cores were tested in compression to determine the development of strength of the concrete. The companion field-cured and standard-cured test cylinders were also tested at the same ages to obtain comparative compressive strengths.

Eighty each of 6 x 12-in. (15 x 30-cm) cylinders, 4 x 8-in. (10 x 20-cm) cylinders, and 4 x 8-in. (10 x 20-cm) drilled cores were tested.

PREPARING AND CURING OF TEST SPECIMENS

The investigation was started in early December 1970 and was terminated late in July 1971 after almost 8 months. For each test series, a 1.5-yd³ (1.15-m³) batch of concrete was ordered from a local ready-mix concrete supplier. The columns/prisms and slabs were cast on a specially built platform erected in an exposure plot situated at the University of Ottawa, Ottawa, Ontario (Figure 1). The large specimens were compacted by an internal vibrator.

For Mix Series I, II, and III, the 6 x 12-in. (15 x 30-cm) and 4 x 8-in. (10 x 20-cm) cylinders were also cast at this exposure site using procedures outlined in ASTM Standard C 31-69. Immediately after casting, the platform was enclosed* by tarpaulins (Figure 1 b) and propane gas heaters were ignited inside the enclosure to bring the ambient temperature to $50 \pm 10^{\circ}\text{F}$ ($10 \pm 6^{\circ}\text{C}$). This temperature was maintained for the next 3 days. The history of the ambient temperature for the exposed specimens is described in the next section of this paper. For concrete Mix Series IV and V, sufficient concrete was taken from the field to the Civil Engineering Materials Behaviour Laboratory at the University of Ottawa, a short distance away, where the 6 x 12-in. (15 x 30-cm) and 4 x 8-in. (10 x 20-cm) cylinders were cast. This procedure was adopted to ensure that the small specimens would not be frozen in case of break-down of the propane-gas heaters.

All cylindrical specimens were compacted by hand rodding; the 6 x 12-in. (15 x 30-cm) cylinders were compacted in three equal layers and the 4 x 8-in. (10 x 20-cm) cylinders were compacted in two equal layers using standard methods. For any particular mix, it was ensured that the same operator carried out the hand rodding of the specimens.

*Provision was made for venting the enclosure.

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For Mix Series I, II, III, all test specimens were left in the heated enclosure for the first 24 hours, following which the eight 6 x 12-in. (15 x 30-cm) and eight 4 x 8-in. (10 x 20-cm) cylinders were removed to the moist-curing room for curing at $73.4 \pm 3^{\circ}\text{F}$ ($23 \pm 1.7^{\circ}\text{C}$) and 100 per cent relative humidity. For Mix Series IV and V, all the test specimens were covered with wet burlap immediately after casting and were left in the laboratory for the next 24 hours. Following this, eight 6 x 12-in. (15 x 30-cm) and eight 4 x 8-in. (10 x 20-cm) cylinders were removed to the moist-curing room for standard curing while the remaining eight 6 x 12-in. (15 x 30-cm) and eight 4 x 8-in. (10 x 20-cm) cylinders were transferred to the exposure plot and placed inside the heated enclosure for field curing.

Immediately before casting, the temperature, slump, unit weight, and air content of the fresh concrete were determined using ASTM Standard methods. The physical properties and chemical analyses of the normal portland cement (ASTM Type 1) used for the concrete mixes are given in Table 1. The mix proportions were supplied by the ready-mix concrete supplier, and the properties of the fresh concrete are shown in Table 2.

AIR TEMPERATURE VARIATIONS AT EXPOSURE STATION

As previously noted, each of the five series of exposed specimens consisted of a column (Mixes I, II, and III), a horizontal prism (Mixes IV and V), a slab, and eight of each of 6 x 12-in. (15 x 30-cm) and 4 x 8-in. (10 x 20-cm) cylinders.

A continuous record of the ambient air temperatures was made from December 21, 1970 to July 27, 1971.

A hygro-thermograph was placed in the standard Stephenson shelter shown in Figure 2, which was obtained from the Department of Transport, Meteorological Branch, Toronto, Ontario. The shelter was placed a short distance from the platform at the exposure station.

The calibration of the hygro-thermograph was checked for a few weeks, at the start of the tests, and periodically later on. The hygro-thermograph device consisted of a 7-day clock mechanism which rotated a drum on which a chart was fixed. The recording pen was activated by the flexing of a bimetallic arm.

The continuous record of temperatures from the field station was compared to the hourly dry-bulb temperatures recorded by the Meteorological Office, International Airport, Uplands, Ottawa. The accuracy of the continuous record was confirmed by comparing the average daily temperatures computed for these two stations. The difference between these computed average daily temperatures was taken in order to determine any significant relationship. The field station is in the downtown area of Ottawa, and the airport is about ten miles south of it, just outside the city. The airport average temperatures were found to

be consistently lower; its average computed value was 1.8°F lower, and the average range of its variation was from -0.3° to -3.8°F . The average daily air temperatures at the exposure station given by the above analysis have been plotted in Figure 3; the heavy shaded rectangles at the top left indicate the temperature range and duration for each test series.

It was not possible to completely isolate the specimens of earlier castings during the heating stage of later series because space on the exposure platform was restricted. All specimens of Test Series I and II underwent the same heating as the specimens of Series III between 16 and 19 December, 1970. The column specimens for Series I, II, and III were exposed to the ambient temperatures while the remaining specimens of these series underwent the heating of Series IV and V between 21 and 24 December, 1970. With regard to humidity inside the heated enclosure, all test specimens were covered with burlap, kept moist for three days.

TESTING OF CONCRETE SPECIMENS

The laboratory- and field-cured cylinders were tested in pairs at 7*, 14, 28, 91, and 218 days. At each of these ages, two 4 x 8-in. (10 x 20-cm) cores were drilled from each column/prism and slab. The four cores for each series were transported to the Mines Branch laboratory for testing. During the winter months, i.e., for ages of 7, 14, 28, and 91 days, the drilled cores and field-cured cylinders were allowed to thaw in the laboratory air for about 6 hours. It was estimated from the limited laboratory studies available that this would allow the inside of the specimens to reach a minimum temperature of about 60°F (15.5°C). The ends of the cores were sawn to get a reasonably smooth surface. All cores and cylinders were capped with a sulphur-flint mixture, and the compression testing was done on an Amsler testing machine of 600,000-lb (271,800-kg) capacity. It should be emphasized that all cores were tested dry in order to simulate the condition of the concrete in exposed structures. The size of the cores was 4 x 8 in. (10 x 20 cm) for a diameter:length ratio of 2, and no correction factors were needed for core strengths (6).

At the ages of 91 and 218 days, the densities of the standard-cured cylinders and of the cores drilled from slabs and columns/prisms were determined.

A view of the drilling operation is shown in Figure 4. A summary of the test results is given in Tables 3 to 5.

* For Mix Series I and II, testing was done starting at 7 days instead of at 14 days; for the Mix Series III, IV, and V, the testing was started at 14 days.

PRESENTATION OF TEST RESULTS

The typical relationships between age and compressive strength for 4 x 8-in. (10 x 20-cm) standard-cured cylinders are shown in Figure 5. The relationships between age and compressive strength as a percentage of the 28-day strength of 4 x 8-in. (10 x 20-cm) standard-cured cylinders are shown in Figures 6 to 10. In order to avoid size effects in the analysis, the 4 x 8-in. (10 x 20-cm) cylinder was selected for comparison with the drilled cores. However, to facilitate comparison of the strengths of 6 x 12-in. (15 x 30-cm) standard-cured and/or field-cured cylinders and the drilled cores, Figure 11 has been prepared to relate the strengths of the two sizes of cylinders.

To determine the reproducibility of the test results, within-batch standard deviations and coefficients of variation have been computed for the test data. These are shown in Tables 6 and 7.

STRENGTH DEVELOPMENT WITH AGE

Figures 6 to 10 relate the compressive strengths of concrete in the standard-cured cylinders, in columns or prisms, and in slabs. It should be pointed out that, in the Ottawa area, the annual freeze-thaw cycles number about 80. The freeze-thaw phenomena are not expected to affect the performance of the field-cured concrete because all the concrete was air-entrained. The relative humidity during winter months in the Ottawa area generally varies from 65 to 75 per cent, whereas in April, May, and June, the relative humidity varies between 50 and 60 per cent.

Mix Series I

Figure 6 shows the strength gain with age for concrete having a compressive strength of 4025 psi (283 kg/cm²) at 28 days, as measured on 4 x 8-in. (10 x 20-cm) cylinders. The standard-cured cylinders show continuous gain in strength up to 218 days and reach a value of 155.0 per cent of the standard strength*. There is no evidence of a levelling-off of the strength at 218 days, which is unusual. The field-cured cylinders show some gain in strength, between 7 and 28 days, reaching to a value of 34.9 per cent of the standard strength. After 28 days, there is a rapid gain in strength and this continues up to 218 days, though at a slower rate after 91 days, to reach a maximum of 105.5 per cent of the standard strength at 218 days. The cores from the slab and columns show rapid gains in strength up to 28 days, reaching 78.5 and

* Hereafter the 28-day compressive strength of 4 x 8-in. (10 x 20-cm) standard-cured cylinders will be referred to as the standard strength.

91.5 per cent of the standard strength, respectively. It should be noted that the cores from the slabs had 7-day strengths of only 28.4 per cent of the standard strength. Both slab and column cores continue to gain strength after 28 days and the rate of strength gain accelerates after 91 days. At 218 days, the cores just surpass the strengths reached by standard-cured cylinders at that age; the highest strength, 162.5 per cent of the standard strength, was obtained for the cores from the columns. The increased rate of strength gain between 91 and 218 days is due to higher ambient temperatures during this period.

Mix Series II

For the low-strength concrete (Figure 7), the standard-cured cylinders show continuous strength gain up to 91 days after which the compressive strength levels off. The test specimens appear to have reached their ultimate strength of 136.2 per cent of the standard strength at 218 days. The field-cured cylinders and the cores from both columns and slabs show rapid gain in strength up to 28 days, although lagging behind the standard-cured cylinders. Between 28 and 91 days, the strength gain for these specimens continues less rapidly. After 91 days, during which there is a change in curing conditions due to changed environmental temperatures, the cores from both slabs and columns show accelerated strength gain. The cores from the columns surpass the strength of standard-cured cylinders but those from the slabs barely reach the standard strength; core strength, 188.7 per cent of the standard strength at 218 days, appears too high. The field-cured cylinders also show somewhat accelerated rate of strength gain but, at 218 days, reach a value of only 97.5 per cent of the standard strength.

Mix Series III

The strength-age relationship for Mix Series III [compressive strength of 4430 psi (310 kg/cm²) at 28 days] is shown in Figure 8. The standard-cured cylinders show a steady gain in strength up to 218 days after which the gain tapers off; 126.0 per cent of the standard strength is reached at this age. The field-cured cylinders gain strength at a slower rate between 14 and 28-days but the strength gain accelerates after 28 days and shows steady increase up to 218 days to reach 103.1 per cent of the standard strength. The slab and column cores show very slow gain in strength between 14 and 28 days but, like the field-cured cylinders, the strength gain accelerates after 28 days and continues until 218 days. Compressive strength values of 114 and 105 per cent of the standard strength are reached at 218 days by the cores drilled from slabs and columns, respectively. There appears to be no change in rate of gain in strength after 91 days as is the case with Mix Series I and II. This is unexplained.

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Mix Series IV

Figure 9 shows the age-strength relations for Mix Series IV with 28-day compressive strength of 5045 psi (355 kg/cm²). The standard-cured cylinders show steady increase in strength to 120.5 per cent of the standard strength at 217 days. The field-cured cylinders show almost no gain in strength between the ages of 14 and 28 days. This is followed by a rapid gain in strength and 95.2 per cent of the standard strength is reached at 91 days. The two 4 x 8-in. (10 x 20-cm) cylinders for testing at 217 days were lost at the outdoor exposure plot, hence no strength values are available for this age.

The cores drilled from the slabs and prisms show steady gain in strength to 95.5 and 88.0 per cent of the standard strength, respectively, up to 91 days. After 91 days, the cores drilled from prisms show accelerated gain in strength to 115 per cent of the standard strength at 217 days. However, the cores from the slabs gain less rapidly in strength reaching 101 per cent of the standard strength at 217 days.

Mix Series V

Figure 10 shows the development of strength with age for Mix Series V with 28-day compressive strength of 6215 psi (437 kg/cm²). As expected, the standard-cured cylinders show continuous gain to 108 per cent of the standard strength at 217 days. The field-cured cylinders show a rapid gain in strength between 28 and 91 days and almost reach the standard strength at 91 days. However, after this, there is a slight drop in strength but 94.5 per cent of the standard strength is reached at 217 days; the drop in strength is unusual. The cores drilled from both slab and prisms show modest gain in strength up to 91 days. After this, the rate of gain in strength accelerates. The cores drilled from the prisms reach 111.1 per cent of the standard strength at 217 days. This is a few percentage points higher than reached by the standard-cured cylinders. The cores drilled from slabs follow the cores from the prism reaching a value of 107.0 per cent of the standard strength at 217 days. In fact, it could be stated that in this series the strengths of the cores at 217 days are of the same order as those reached by the standard-cured cylinders.

DENSITIES OF TEST CYLINDERS AND CORES

The 28- and 91-day densities of 4 x 8-in. standard-cured cylinders and drilled cores from columns and slabs are given in Table 5. The densities of the cylinders vary from 137.33 to 151.03 lb/ft³ (2200 to 2420 kg/m³), but those of cores vary from 134.22 to 150.04 lb/ft³ (2150 to 2404 kg/m³); the cylinders had been compacted by hand-rodding,

whereas the slabs and columns had been compacted by an internal vibrator. There appear to be no significant differences between the densities of cores drilled from columns and those drilled from slabs.

REPRODUCIBILITY OF TEST RESULTS

The within-batch standard deviation and coefficient of variation of the test results at 28 and 91 days are shown in Tables 6 and 7. The analyses indicate that at 28 days the field-cured cylinders have the poorest reproducibility with an average C.V. value of 10.7 per cent; the corresponding value for the standard-cured cylinders is 7.5 per cent. The C.V. values for the compressive strength of drilled cores from slabs and columns are 6.9 and 4.8 per cent respectively.

At 91 days, the within-batch C.V. for compressive strengths of field-cured cylinders shows marked decrease whereas those for the standard-cured cylinders and the drilled cores are of the same order as at 28 days. No positive conclusions can be drawn for the within-batch variation for 218-day test results because too few results are available.

STRENGTH DEVELOPMENT AND MATURITY CONCEPT

The combined effect of time and temperature is generally referred to as maturity (7). Often, this concept has been correlated with compressive strength, particularly if the curing history of the specimen changes or if the strengths of concrete specimens, cured under two different curing regimes, are being compared. An attempt was made to see if this concept could explain the strength gain of the concrete under investigation. It is seen from the data in Table 3 that at all ages, the strength of field-cured cylinders is considerably lower than the strength of cores drilled from the large specimens. Specifically, at 218 days, the difference in strength varies from a low of 475 psi (33 kg/cm²) for Mix Series III to a high of 2250 psi (158 kg/cm²) for Mix Series I. Again, the cores drilled from slabs have generally lower strengths than those from the prisms or columns, though the strength difference is much less marked. This illustrates that, for the maturity concept to be applicable to large concrete specimens cured in the field, allowance must be made for their size because of different temperature conditions inside the large concrete specimen. Furthermore, maturity calculations indicate that at low temperatures the maturity rule considerably under estimates the potential strength of concrete. Unfortunately, because of poor working conditions at the exposure station, it was not possible to install thermocouples inside the large test specimens.

RELATION BETWEEN THE COMPRESSIVE STRENGTHS OF DIFFERENT-SIZED SPECIMENS

The relationships shown in Figures 6 to 10 are for cylinders and drilled cores having a diameter of 4 in. (10 cm) and length/diameter ratio of 2. The above size was selected in order to have direct comparison between the compressive strength of the drilled cores and the standard- and field-cured cylinders. However, 6 x 12-in. (15 x 30-cm) cylinders were also cast and tested for both standard- and field-curing conditions. Figure 11 shows this relationship between the 28-day compressive strength of the different-sized cylinders. There is an indication that the difference in the strength of cylinders, 4 and 6 in. (10 and 15 cm) in diameter, increases with increase in the strength level of the concrete, the strength of 6-in. (15-cm) cylinders being generally lower than the 4-in. (10-cm) cylinders.

CONCLUDING REMARKS

Concrete, initially cured for three days at about 50°F (10°C) and 75 per cent relative humidity* and then exposed to below-freezing temperatures, continues to gain strength. The gain in compressive strength is considerably more rapid by column and slab concrete than by field-cured concrete cylinders.

The rate of gain in strength of field-cured concrete shows a marked increase after the ambient temperature of exposure exceeds 50°F (10°C). Concrete cured below freezing has a reserve potential strength which is realized when the ambient air temperature and humidity conditions approach the conditions for standard moist-curing.

At the age of 7 months, the compressive strengths of the cores drilled from columns and slabs approach or exceed the compressive strengths of test cylinders cured under standard moist conditions.

At temperatures below 30°F (-1°C), the concept of maturity cannot be applied to estimate the compressive strength of concrete because the maturity rule considerably under estimates the potential strength of concrete and, for the same maturity, different-sized specimens have different strengths.

RECOMMENDATIONS

It is recommended that additional field data be obtained to confirm the findings reported in this paper. Further, it is suggested

*Estimated value only. All specimens were covered with wet burlap kept moist for the duration of three days.

that investigations be performed to determine the compressive strength of field-cured test specimens at below-freezing temperatures after initial curing of 1, 2, 3, 5, and 7 days at 55°F (12.8°C) and at 70 to 100 per cent relative humidity.

REFERENCES

1. Lapinas, R.A.; "Strength Development of High Cement Content Concrete Cast in Large Sections", presented at Fall Meeting, American Concrete Institute, Toronto, November, 1963.
2. Peterson, N.; "Strength of Concrete in Finished Structures", Transactions No. 232, Swedish Cement and Concrete Research Institute, Royal Institute of Technology, Stockholm, 1964.
3. Bloem, D.L.; "Concrete Strength Measurement - Cores vs Cylinders", Proceedings, ASTM, Philadelphia, V. 65, 1965, pp. 668-696.
4. Bloem, D.L.; "Concrete Strength in Structures", Proceedings, Journal American Concrete Institute, Vol. 65, No. 3, March 1968, pp 176-187.
5. Henzel, J.; and Grube, H.; "Strength Studies of Concrete on an Actual Job and of the Corresponding Control Cubes", Der Bauingenieur, Vol. 41, No. 12, pp 487-491, 1966 (NRC Translation No. 1327).
6. ASTM Standard C 42-68: Obtaining and Testing Drilled Cores and Sawed Beams of Concrete. ASTM Book of Standards, Part 10, 1970.
7. Malhotra, V.M.; "Maturity Concept and the Estimation of Concrete Strength - A Review", Mines Branch Information Circular IC 277, November 1971, 43 pp.

TABLE 1

Physical Properties and Chemical Analysis of Cement*

Description of Test	
Time of Set (Vicat needle): Initial	3 hr 55 min
Final	- -
Specific Surface (Blaine)	3245 cm ² /g
Soundness - Autoclave	0.12
<u>Physical Tests - Mortar Strength</u>	
Compressive Strength of 2-in. (5-cm) cubes	
3-day	2810 psi (197 kg/cm ²)
7-day	3900 psi (274 kg/cm ²)
28-day	5190 psi (365 kg/cm ²)
<u>Chemical Analysis</u>	
Insoluble Residue	0.22 per cent
Tricalcium Silicate (C ₃ S)	51.1 per cent
Tricalcium Aluminate (C ₃ A)	9.9 per cent
Magnesium Oxide (MgO)	3.50 per cent
Sulphur Trioxide (SO ₃)	2.61 per cent
Loss on Ignition	0.65 per cent

*Test results and chemical analyses supplied by the cement manufacturing company.

TABLE 2

Mix Design Data and Properties of Fresh Concrete

Mix No.	Mix Design Data*				Properties of Fresh Concrete						
	Nominal Water/Cement Ratio by Weight	Cement Content		Aggregate/Cement Ratio by Weight	Temperature		Slump,		Unit Weight,		Air Content, Per Cent
		lb/yd ³	kg/m ³		°F	°C	in.	cm	lb/ft ³	kg/m ³	
I	0.67	410	243	8.25	64	18	2.0	5.1	148.4	2377	3.8
II	0.77	360	214	9.46	50	10	2.5	6.4	141.6	2268	3.6
III	0.56	500	297	6.76	62	17	**	**	133.2	2134	9.5
IV	0.46	630	374	5.06	52	11	5.0	12.7	140.4	2249	7.0
V	0.42	700	415	4.39	64	18	3.5	8.9	145.6	2333	5.0

*Supplied by the ready-mixed company which delivered the concrete.

**Slump exceeded 6 in. (15 cm).

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TABLE 3
Compressive Strength of Standard- and Field-Cured Cylinders and of
Cores Drilled from Slabs, Beams, and Columns

Mix No.	Compressive Strength at Various Ages of 4 x 8 in. (10 x 20-cm) Cylinders or Cores, psi (kg/cm ²)															
	7-day				14-day				28-day				91-day			
	Standard Cured	Field Cured	Slab Cores	Column Cores	Standard Cured	Field Cured	Slab Cores	Column Cores	Standard Cured	Field Cured	Slab Cores	Column Cores	Standard Cured	Field Cured	Slab Cores	Column Cores
I	3405 (186)	1105 (78)	1140 (80)	2695 (189)	-	-	-	-	4025 (283)	1415 (99)	3155 (222)	3670 (258)	5370 (378)	3425 (241)	4960 (349)	5050 (355)
II	1680 (118)	710 (50)	465 (33)	1330 (94)	-	-	-	-	1895 (133)	1460 (103)	1280 (90)	1605 (113)	2440 (172)	1640 (115)	1450 (102)	1760 (124)
III	-	-	-	-	3735 (263)	1395 (98)	2760 (194)	2110 (148)	4430 (311)	1770 (124)	2850 (200)	2140 (150)	5310 (373)	3495 (246)	3935 (277)	3585 (252)
IV	-	-	-	-	4520 (318)	3015 (212)	3640 (242)	3810 (268)	5045 (355)	2990 (210)	3965 (279)	4095 (288)	na	4780 (336)	4805 (388)	4625 (311)
V	-	-	-	-	5540 (390)	4095 (288)	3640 (256)	4250 (299)	6215 (437)	3420 (240)	4325 (304)	4620 (325)	na	6180 (435)	5040 (354)	5685 (400)

*Only one cylinder available for testing.
 **Test cylinders were lost.
 ***Test results not available.
 +For Mix Series IV and V, age at testing was 217 days.
 +Strength estimated from 6 x 12-in. (15 x 30-cm) cylinders which had strength of 2545 psi.

TABLE 4
Compressive Strength of Standard- and Field-Cured Test Cylinders of Different Sizes

Mix No.	Curing Condition	7-day		14-day		28-day		91-day		218-day*	
		6x12-in. (15x30-cm)	4x8-in. (10x20-cm)	6x12-in. (15x30-cm)	4x8-in. (10x20-cm)	6x12-in. (15x30-cm)	4x8-in. (10x20-cm)	6x12-in. (15x30-cm)	4x8-in. (10x20-cm)	6x12-in. (15x30-cm)	4x8-in. (10x20-cm)
I	Standard	3920 (276)	3405** (239)	-	-	4660 (328)	4025 (283)	5300 (373)	5370 (378)	5550 (390)	6220** (437)
	Field	1570 (110)	1105 (78)	-	-	2000 (141)	+	+	3425** (241)	+	4260 (299)
II	Standard	1905 (134)	1680 (118)	-	-	2265 (159)	1895 (133)	2550 (179)	2440 (172)	2545 (179)	+
	Field	475 (34)	710 (50)	-	-	1140 (80)	1460 (103)	1010 (71)	1640 (115)	2475 (174)	+
III	Standard	-	-	3655 (257)	3735 (263)	4110 (289)	4430 (311)	4320 (304)	5310 (373)	4880 (343)	5580 (392)
	Field	-	-	1225 (86)	1395 (98)	1565 (110)	1770 (124)	2780 (195)	3495 (246)	4305 (303)	4565 (321)
IV	Standard	-	-	3995 (281)	4520 (318)	4100 (286)	5045 (355)	5130 (361)	+	5200 (366)	6055 (426)
	Field	-	-	2740 (193)	3015 (212)	2355 (166)	2990 (210)	4150 (292)	4780 (336)	5625 (395)	+
V	Standard	-	-	4965 (349)	5540* (389)	5565 (391)	6215 (437)	6350 (446)	+	7000 (492)	6710 (472)
	Field	-	-	3855 (271)	4095** (288)	3330 (234)	3420 (240)	5200 (366)	6180 (435)	6820 (479)	5890** (414)

*For Mix Series IV and V, age at testing was 217 days.
 **Only one cylinder available for testing.
 +Test results not available.