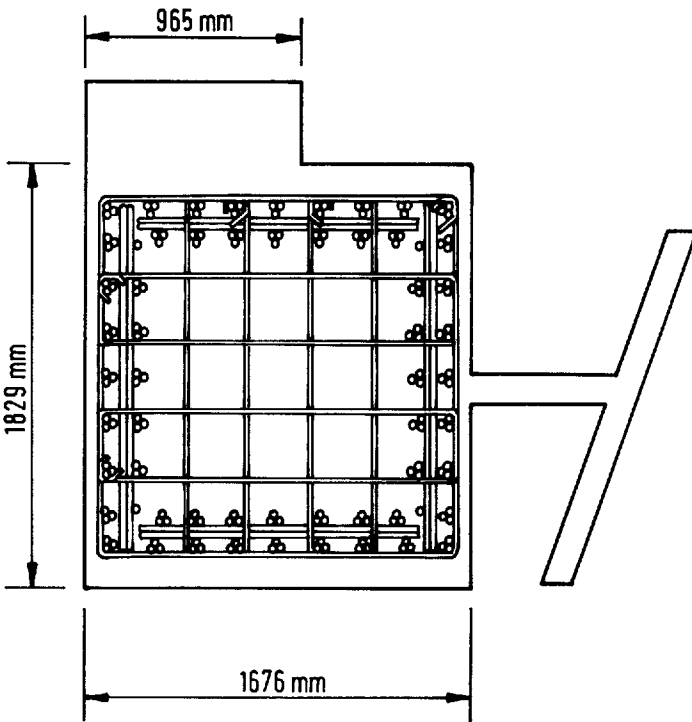
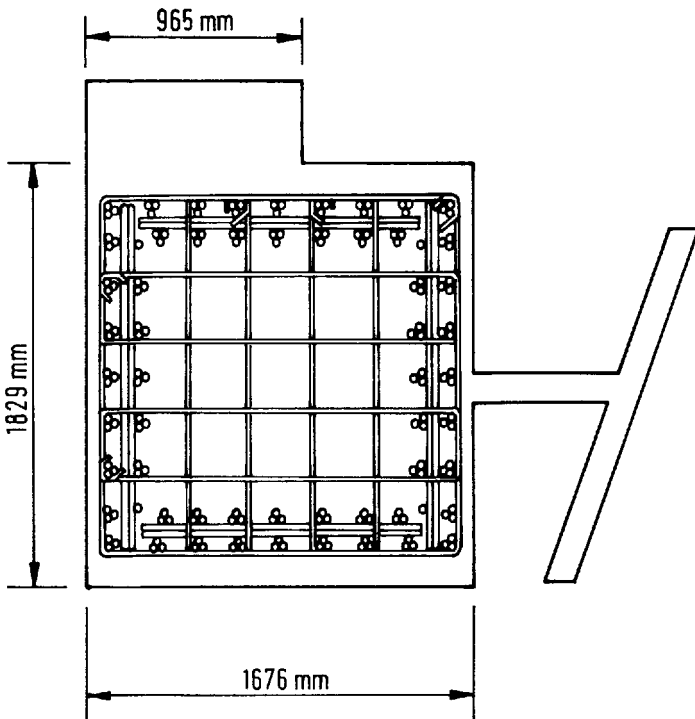


Durable Concrete in Hot Climates



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Durable Concrete in Hot Climates



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PREFACE

The International Symposium on "How to Produce Durable Concrete in Hot Climates," sponsored by Committee 201 on Durability of Concrete, was held at the ACI Fall Convention in San Juan, Puerto Rico in October 1992.

Altogether, ten papers were presented at the two sessions of the symposium. Approximately seventy-five persons were in attendance at each session.

The symposium was noted for its international flavor and the variety of topics presented. These included both field and laboratory studies and covered such subjects as high temperature effects on the microstructure of concrete; various topics relating to the corrosion of steel in reinforced concrete; concreting problems associated with hot-dry, hot-wet and marine environments; concreting problems associated with the placement of thick-sections; and the effect of various curing conditions on strength development.

I would like to thank Messrs. Gunnar Idorn and Ignacio Martin who, along with me, comprised the subcommittee that organized this symposium. I also wish to thank the many members of Committee 201 who helped review the manuscripts which make up this volume.

Cameron MacInnis
Editor

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Effect of Hot Weather Conditions on the Microcracking and Corrosion Cracking Potential of Reinforced Concrete

by Rasheeduzzafar and S.M.A. Al-Kurdi

Synopsis: This study focuses on the damaging implications of the daily temperature fluctuations in the aggressive climatic conditions of hot-arid regions due to strain incompatibility resulting from the widely differing coefficients of thermal expansion of the local crushed limestone aggregate and the hardened cement paste. The data developed strongly indicates that temperature fluctuations cause microcracking in concrete which would increase its permeability and lower its tensile strength and cracking time to a noticeable degree. In this investigation, concrete specimens of water-cement ratios 0.40, 0.50, and 0.65 with a cement content of 550 lb/yd³ (330 kg/m³) were subjected to cyclic heating in programmed ovens which carried out 120 temperature fluctuations, each simulating the temperature regime of a typical summer day in Eastern Saudi Arabia. The thermal regime was characterized by a temperature swing from 27 °C to 60 °C within a 24 hr period. This included the effect of concrete surface heating by direct solar radiation. Pulse velocity, permeability, and time-to-cracking data were developed in reference and cyclic heat-treated specimens at 20, 40, 60, 80, and 120 heating cycles. It was observed that the cyclic heat treated specimens had a significantly reduced pulse velocity, a noticeably increased permeability, and, depending on water-cement ratio, a 55 to 70 percent reduction in cracking time due to reinforcing bar corrosion. It is inferred from this data that a significant degree of microcracking is induced in concrete due to the thermal incompatibility of concrete components.

Microcracking of concrete increased with the number of heating cycles and there was a systematic improvement in the performance of concrete with a reduction in w-c ratio.

Keywords: Corrosion; cracking (fracturing); cyclic heat; hot weather construction; microcracking; permeability; reinforced concrete; strains; temperature; tensile strength; thermal expansion; velocity; water cement ratio

2 Rasheeduzzafar and Al-Kurdi

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INTRODUCTION

The exposure conditions for concrete in the Middle East constitute one of the most aggressive climatic environments in the world. In the Arabian Gulf States, interaction between concrete and the environment controls material performance (1-4) and has resulted in premature deterioration and low durability of concrete construction to a level of major concern.

In terms of its sensitivity to its service environment, concrete has been likened to a living organism. Contrary to its plain rock-like appearance and deceptive simplicity, concrete is a highly complex diphasic or three-component (cement, water and aggregate) pseudo-solid system which is affected by many external actions such as variation of pressure, temperature, or humidity. In the Gulf States, many problems of concrete durability have been encountered which are related to the adverse effects of climatic conditions during mixing, placing, curing and the service life of structures.

EFFECT OF HOT WEATHER CONDITIONS ON FRESH AND HARDENED CONCRETE

Climatic conditions in the Gulf region may cause increased water demand and slump loss, premature setting resulting in cold joints and insufficient compaction and enhanced tendency for plastic shrinkage and pre-setting cracking in fresh concrete. Concrete cast and cured under hot weather conditions and not cured sufficiently thereafter may show as much as 25%

reduction in strength (5). Any deficiency in curing which has a negligible effect in mild climatic conditions may permanently impair the quality of concrete skin in hot weather conditions, thereby weakening the protective shield against the ingress of chlorides, sulfates, moisture and oxygen into the concrete matrix. High rates of evaporation of mix and curing water cause significant sulfate and chloride salt concentrations near the surface and may result in sulfate attack and early initiation of rebar corrosion if steel is located near the surface. Thermal and drying shrinkage cracking are a common feature of concrete damage in this region. Corrosion and solubility of salts are accelerated with an elevation of temperature which on the surface of the concrete may be as high as 70 to 80°C due to the direct solar radiation effect on a typical summer day. Fluctuations of temperature cause cycles of expansion/contraction and hydration/dehydration resulting in microcracking and damage due to thermal incompatibility of concrete components.

All these effects interact concomitantly and cumulatively and significantly increase concrete permeability and reduce its durability.

TEMPERATURE FLUCTUATIONS AND MICROCRACKING IN CONCRETE

It is being increasingly recognized that the aggregate-paste bond strength forms the weakest link in the heterogeneous concrete system (6-8). Studies focusing on direct observations of microcracking in concrete show that microcracks can be divided into two categories: bond cracks which exist at the aggregate-paste interface, and mortar cracks which exist within the mortar. These studies also show that considerable microcracking at the aggregate-paste interface exists in concrete even before it is subjected to any external load. Interfacial microcracking is partly due to segregation and partly due to volume changes during hydration and setting. When microcracking is confined to the transition zone, as happens before loading, it is essentially discontinuous. However, when concrete is subjected to external load and thermal/humidity gradients, specially to cycles of temperature and humidity changes, then mortar cracks start bridging and spanning between bond cracks, thereby making the microcracking system within concrete continuous (9). Such a continuous system of microcracks in concrete gives rise to distinct paths and conduits for the ingress of aggressive ions from outside and will play a decisive role in the durability performance of concrete. Also, the existence of considerable microcracks in concrete due to segregation, volume changes during hydration, and setting and cyclic thermal/humidity changes will lower its tensile strength. This will advance cracking failure due to reinforcement corrosion which is caused by the tensile mechanical pressure from the expansive corrosion products.

One aspect of the hot weather condition which has not been adequately researched and yet may cause considerable physical disintegration and microcracking is the large fluctuation in the daily and seasonal temperature conditions. For example, on the coastal flats of the Gulf region the temperature easily varies as much as 20°C during a typical summer's day. This rapid and continuous variation in the temperature causes continual expansion and contraction which may result in considerable microcracking. The expansion-contraction cycle becomes all the more damaging due to the thermal incompatibility of concrete components. The differential expansion and contraction movements of aggregate material and the hardened cement paste may set up tensile stresses far beyond the tensile strength of concrete resulting in microcracking. Limestone, the predominantly used aggregate, is exactly the rock type having the widest spectrum of coefficient of thermal expansion values (1 to $10 \times 10^{-6}/^{\circ}\text{C}$). The coefficient for hardened cement paste is much higher (usually between 10×10^{-6} and $20 \times 10^{-6}/^{\circ}\text{C}$). With the fall of temperature tensile and compressive stresses are set up in the cement paste and the aggregates respectively. With a rise of temperature the stresses are not exactly reversed but tensile stresses are set up at the aggregate-paste interface tending to cause interface bond failure and significant microcracking in and around the transition zone (10). It has been shown by Hsu(11) that a volume change of 0.3% is enough to generate tensile stresses of the order of 1800 lb/in^2 at the aggregate-paste interface. Slate and Matheus (12) have determined volume changes of cement paste and concrete from the time of casting to an age of 7 days. This work shows that volume changes even larger than 0.3% occur during setting and hardening of concrete. The authors, on the basis of a simple mathematical model, have inferred that for a commonly occurring value of $6 \times 10^{-6}/^{\circ}\text{C}$ for limestone, tensile stresses of more than 250 lb/in^2 for every 10°C fall of temperature are set up in the concrete. These stresses are cumulatively interactive on the shrinkage and microfissure stresses already present due to other causes.

RATIONALE FOR THIS STUDY

This study focuses on the effect of cyclic thermal changes on the microcracking in concrete and the lowering of its tensile strength which may significantly advance corrosion cracking. The thermal changes imposed on concrete simulate the daily fluctuations of temperature typical of the ambient conditions during summer months in the Gulf region. Increase in microcracking due to thermal cycles is determined indirectly by two techniques: firstly, by the additional water absorbed under pressure by cyclic heat-treated specimens and secondly, by the changes in the pulse velocity measurements. In this study the lowering of the tensile strength is measured directly by recording the reinforcement corrosion cracking time in accelerated corrosion tests on reference and heat-treated specimens.

EXPERIMENTAL PROGRAM AND TECHNIQUES

Materials and Specimen Preparation

Investigation on microcracking due to cyclic heating was carried out on concretes made with w-c ratios of 0.40, 0.50 and 0.65. Accelerated corrosion tests to evaluate the effect of cyclic heating on corrosion cracking time were carried out on two concretes having w-c ratios of 0.40 and 0.65. All mixtures were made with a cement content of 550 lbs/yd³ and a coarse to fine aggregate ratio of 2:1. The coarse aggregate used was local crushed limestone with 3/4 in. maximum size having a bulk specific gravity of 2.37 and average absorption of 2.3%. The fine aggregate was beach sand with a specific gravity of 2.70 and absorption of 0.27%. The beach sand had a grading significantly finer than the ASTM C 33 limit for sand. Both coarse and fine aggregates were washed free of dust with sweet water which had combined Cl⁻ and SO₄²⁻ less than 100 mg/l. After washing the aggregates were air dried at 110 °C for a period of 20-24 hours. The aggregates were air cooled before use. Desalinated water was used for mixing.

Concrete was mixed in a 1/2 ton electrically operated revolving drum type mixer. The molds were filled in three equal layers and vibrated at a rate of 3600 vibrations per second till complete consolidation was assumed when a uniform thin film of mortar appeared on the concrete surface. After casting, the specimens were kept in air for 24 hours and then demolded and stored in water for 28 days. Thereafter the test specimens were transferred to the programmed ovens for cyclic heating whereas reference specimens were retained in water.

All specimens were 4-inch cubes. In order to evaluate the effect of microcracking on concrete resistance to corrosion cracking, 4 inch cubes were prepared with 1/2 in diameter mild steel bar embedded centrally with a clear cover of 1 inch from the bottom. The bar protruded 2.5 inches out of the concrete cube.

Simulation of Daily Temperature Fluctuations

In order to provide a realistic simulation of daily temperature fluctuations, the actual ambient temperature variation in Dhahran for the six summer months of April to September were analyzed for a 7-year period and are shown in Fig. 1. It is seen that the daily temperature variation is very similar for the summer months of June, July and August. Therefore, an average temperature variation of these three months over a 7-year period, shown in Fig. 2, was adopted as a possible regime to

simulate the daily fluctuations of temperature during the peak summer months. To the maximum ambient temperature must be added the effect of direct solar radiation which raises the temperature of concrete surfaces significantly in excess of the ambient temperature.

An experiment was performed to evaluate the effect of solar radiation on typical concrete surfaces. On several days in the month of July, ambient and concrete surface temperatures were recorded concurrently at hourly intervals from 10 am to 2.30 pm. Based on this test and on data developed on solar radiation effect by Mironov et al (13) and Jaegermann et al (14), a 20 °C addition was made to the maximum ambient temperature for a 4 hour period from 11 am to 3 pm to take into account the effect of direct solar radiation. The somewhat idealized regime adopted for simulating daily temperature fluctuations for summer months is shown in Fig. 3.

The heating and the electrical systems of two ovens were then modified to automatically trace the proposed 24-hour heating cycle. About forty-two 4-inch cubes and steel embedded specimens were then located in each oven in a manner to allow adequate air circulation within the oven. Temperatures were repeatedly checked at different locations within the oven and were found to be fairly uniform in the whole range of the regime. These ovens performed cyclic heating with excellent precision in accordance with the proposed temperature regime of Fig. 3. Fig. 4 shows the ovens with specimens and with mechanical and electronic fixtures to automatically control the cyclic heating. A strip chart recorder was hooked to the ovens to monitor the heating cycle continuously.

The compressive strength, pulse velocity and permeability data developed in this study are up to 120 cycles of heating after a 28 days' curing period for the specimens. Parallel data have been obtained for reference specimens which were kept under water for 120 days after a 28 day curing period.

Permeability Testing

In this study the objective of permeability determination is to develop a relative measure of the degree of microcracking which occurs in concrete as a result of cyclic heating. With increased microcracking the permeability would be enhanced thereby reducing the barrier-effect against the ingress of chloride ions and oxygen to the steel-concrete interface. This will reduce concrete protection against corrosion. There is therefore an important relationship between permeability caused by microcracking and concrete durability.

A special apparatus shown in Fig. 5 was developed and fabricated for high pressure permeability testing which is similar to those developed by Lindsay [15] and Tyler and Erlin [16]. It consists essentially of a water-tight pressure vessel for two 3/4 x 4 inch cylindrical test specimens, a hand pump for applying hydraulic pressure, high quality pressure gauge for measuring hydraulic head, and a graduated glass cylinder for measuring the amount of water pumped into the pressure vessel. The pressure chamber is just large enough to accommodate two 3/4 x 4 inch cylindrical specimens, thus keeping the volume not occupied by a specimen to a minimum. Water flows by gravity from the graduated glass cylinder into the hand pump.

The two 3/4 x 4 inch cylindrical specimens used for the high pressure permeability tests were cored from the reference and cyclic heat treated 4 inch cubes. The reference and cyclic heat-treated cored specimens were brought to a similar state of internal dryness by oven drying at 80° C till the reduction in weight due to loss of water ceased to occur. The cylindrical concrete specimen was then placed into the water-filled pressure vessel and a hydraulic pressure of 1000 psi (6.9 N/mm²) was applied through a hand pump. The rate and the total amount of water forced into the specimen was recorded. The method provides a useful comparison of the relative permeabilities of various concretes used in this program. The apparatus and the procedure are fully described elsewhere (17). The water absorption data developed are based on averages of three specimens for each mixture.

Pulse Velocity Measurements

Proprietary Pundit equipment was used with 82 KHZ transducers coupled to the opposite faces of the cubes with the help of a sticking grease to obtain pulse velocity readings along all three axes. Care was exercised in ensuring good acoustic coupling between transducer face and the surface of the cube.

Resistance to Concrete Cracking

This test was designed to evaluate the effect of microcracking on the resistance against cracking caused by the expansive mechanical pressure generated due to the growth of rebar corrosion products. Specimens with w-c ratios of 0.40 and 0.65 and treated with 164 simulated cycles of daily temperature fluctuations after 28 days of curing were subjected to this accelerated corrosion test where time taken to first corrosion cracking was determined. Concrete with increased microcracking as a result of cyclic heating should crack earlier.

For this test 1/2 inch diameter steel bars embedded in concrete specimens of the type shown in Fig. 6 were used. These specimens were sawn to the 2 x 4 x 4 inch size from the steel-embedded 4 inch reference and cyclic heat-treated cubes (Fig. 7). The proposed size of the specimen ensured that the crack due to rebar corrosion would always occur vertically along the bar on the 4 x 4 inch faces of the specimens. To simulate the corrosion process, direct currents of 20 and 15 milliamperes were impressed on the bar embedded in the cube using a system incorporating a small rectifier power supply coupled with an ammeter to monitor the current, and a potentiometer to control the current intensity. Each specimen was placed on plastic chairs under water in a galvanized steel tank. The direction of the current was arranged so that the reinforcing steel served as the anode, while the galvanized steel tank body acted as the cathode. To facilitate an even spread of the corrosion current over the bar surface the specimens were soaked in water for at least 2 hours prior to the test. The current supplied to each specimen was checked on a regular basis and any drift was corrected by the adjustment of the potentiometer.

Appearance of the first crack in each specimen was monitored precisely by installing 2 inch SR-4 strain gauges horizontally near the top on the opposite sides of the 4 inch square faces of the specimen. The gauges were fixed very carefully on prepared concrete surfaces and were protected with an epoxy coating to avoid moisture penetration. The gauges were connected to a strain recording system which was hooked to a strip chart recorder to record strain continuously with time. As soon as the crack occurred, the strain gauge was damaged which was instantaneously recorded by the strip chart recorder. The accelerated corrosion test instrumentation is shown in Fig. 8.

Compressive Strength

Compressive strengths of reference and cyclic heat-treated specimens were determined after every 20 days/cycles in accordance with ASTM C 39 with a strain rate of 144 kN/minute.

RESULTS AND DISCUSSION

Results

Fig. 9 shows the effect of cyclic heating on the compressive strength of specimens made with w-c ratios of 0.4, 0.5 and 0.65. The strength data have been developed for 148 days for the reference specimens and for 120 heating cycles for the heat-treated specimens.

Fig. 10 shows the effect of cyclic heating on the pulse velocity measurements for reference and heat-treated specimens made with the three w-c ratios.

The permeability data for the cyclic heat-treated specimens for 120 cycles in comparison with the reference specimens for the three concretes are shown in Fig. 11 Fig. 12 shows a comparison of the water absorption capacities of 0.40 w-c concrete specimens subjected to 80 and 120 heating cycles.

Table 1 shows the effect of cyclic heating on the time taken for the concrete to crack as a result of tension developed due to the mechanical pressure exerted by the corrosion products. The specimens tested in this series were made with two w-c ratios of 0.40 and 0.65.

Discussion

The compressive strength of specimens subjected to cyclic heating increased with the number of heating cycles up to about 60 cycles, after which it almost stabilized or showed slight reduction. Such reduction which in any case has not exceeded 3%, may be considered insignificant for all practical purposes and may be attributed to the unavoidable lack of uniformity associated with concrete making and testing. The increase in compressive strength during the first 60 cycles is significantly higher for the heat-treated specimens compared to reference specimens, and may be attributed to accelerated hydration of cement at higher temperatures.

In spite of the fact that compressive strength has increased during the first 60 heating cycles, it is note-worthy that the structure of concrete has been deteriorating during this period due to the daily fluctuations of temperature. This is concomitantly shown by the three different techniques used in this study: firstly, by a progressive reduction in the pulse velocity measurements, secondly, by an increase in the permeability and thirdly, by a reduction in the cracking time of concrete for specimens subjected to cyclic heating.

The pulse velocity data of Fig. 10 for reference and heat-treated specimens up to the age of 148 days show increasing values of pulse velocity with time for reference specimens signifying a densification of the structure of concrete with the progress of hydration. However, at the same time pulse velocity measurements for specimens subjected to cyclic heating show a reduction in pulse velocity values for concretes of all the three w-c ratios, signifying an increase in the microcracking due to the thermal incompatibility of concrete components.

This position is also supported by the water absorption data presented in Fig. 11. It is seen that for each of the three w-c ratios the absorptive capacity, signifying permeability, is higher for cyclic heat-treated specimens than for the parallel reference specimens. Also, the absorptive capacity and hence permeability of specimens subjected to 120 heating cycles is somewhat higher in comparison with those subjected to 80 heating cycles (Fig. 12).

Table 1 shows the time taken for concrete specimens to crack as a result of induced rebar corrosion for reference and cyclic heat-treated specimens made with w-c ratios of 0.40 and 0.65. Time-to-cracking indirectly signifies the tensile strength of concrete. The time-to-cracking data show that the cyclic heat-treated specimens for the two mixes cracked significantly earlier than the reference specimens. The time taken for the cyclic heat-treated specimens to crack was half to one-third of the time taken by the reference specimens of the corresponding mixtures. This shows that the microcracking in the internal structure of concrete lowered its tensile strength thereby advancing its corrosion cracking time.

Another significant observation was the exudation of liquid corrosion products at the bar concrete junction which happened in the accelerated corrosion test for the cyclic heat-treated specimens only. The control specimens did not show the exudation of corrosion products and it is possible that the bars in the heat-treated specimens were undergoing corrosion through a modified mechanism. The exudation of corrosion products appeared to suggest a deterioration of the rebar-concrete bond thereby allowing noticeable exudation of rust products to take place.

The effect of concrete quality on its durability performance is clearly shown by the pulse velocity, permeability and the time-to-cracking data. All the three techniques show a systematic improvement in the performance of concrete in permeability, pulse velocity and time-to-cracking tests with a reduction in the w-c ratio. For example, after the cyclic heat-treatment, the 0.40 w-c concrete specimens took twice as much time for corrosion cracking as the 0.65 w-c concrete specimens.

CONCLUSIONS

1. Pulse velocity, permeability and time-to-cracking data are developed on reference and cyclic heat-treated concrete specimens. These data show that cyclic heating, simulating the fluctuations of temperature of a typical summer day in Eastern

Saudi Arabia, significantly reduces pulse velocity, noticeably enhance permeability, and, depending on w-c ratio, result in a 55 to 70 percent reduction in corrosion cracking time. It is inferred from these data that a significant degree of microcracking is inducted in concrete due to the thermal incompatibility of concrete components.

2. Microcracking of concrete increases with the number of heating cycles and there was a systematic improvement in the performance of concrete in permeability, pulse velocity and time-to-cracking tests with a reduction in w-c ratio. For example, after the cyclic heat-treatment, the 0.40 w-c concrete specimens took twice as much time for corrosion cracking as the 0.65 w-c concrete specimens.

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TABLE 1 — TIME-TO-CRACKING IN ACCELERATED REINFORCEMENT CORROSION TEST

Specimens	Time to cracking, Hours	
	Impressed Current, milli amperes	
	20	15
Reference, w-c=0.40	46	92.5
Cyclic heat-treated w-c = 0.40	20	38.5
Reference, w-c=0.65	34	46.5
Cyclic Heat-treated, w-c=0.65	11	17.0

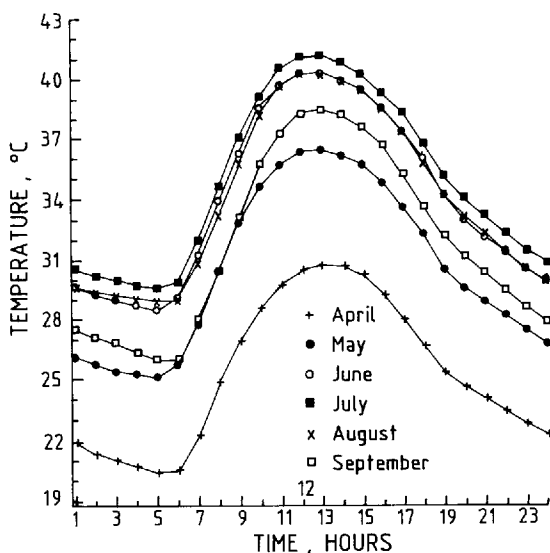


Fig. 1—Average daily temperature variation at Dhahran for six summer months (average of a 7-year period)

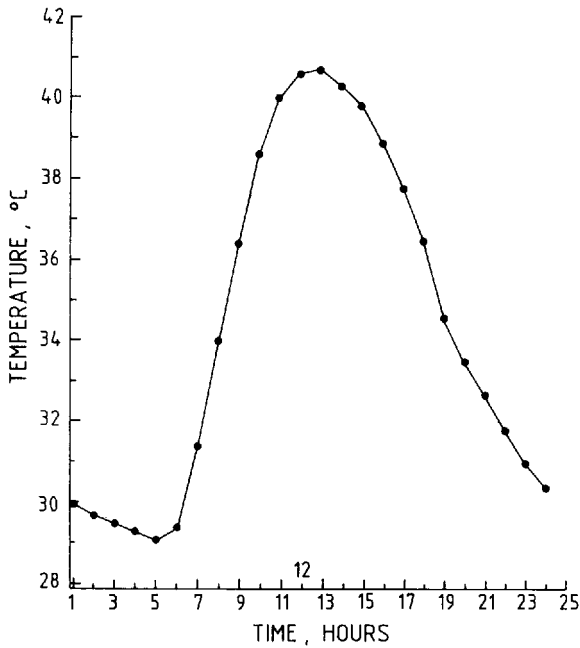


Fig. 2—Typical average daily summer temperature at Dharan (based on an average for the months of June, July, and August)

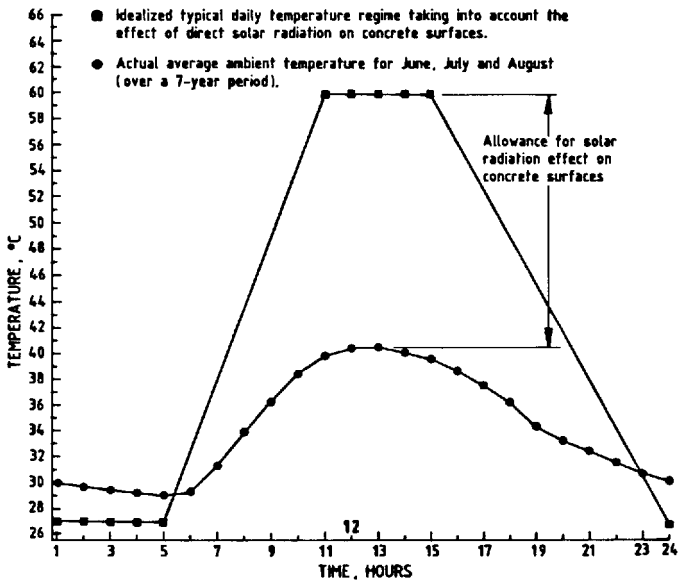


Fig. 3—Actual and idealized typical daily temperature taking into account the effect of direct solar radiation on concrete surfaces

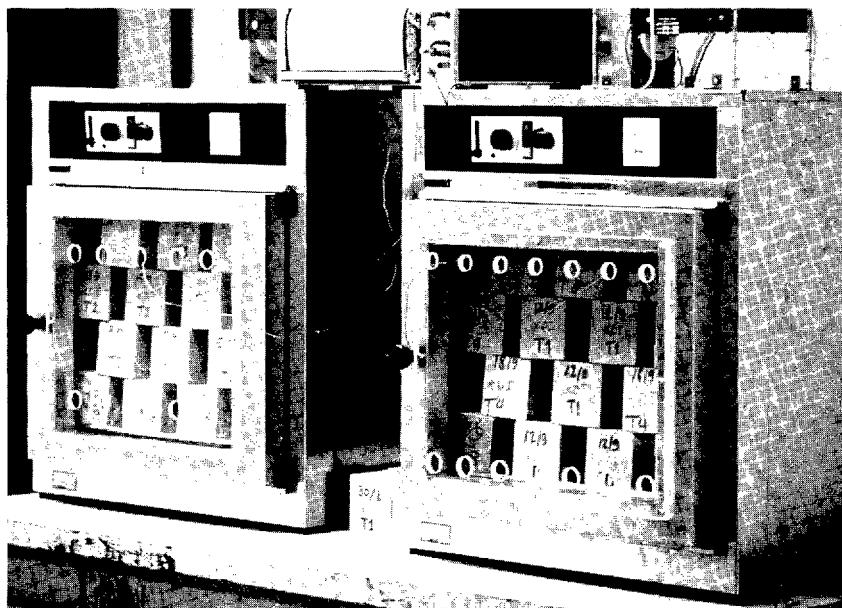


Fig. 4—Specimens positioned in programmed temperature chambers for cyclic heat treatment to simulate diurnal fluctuations of temperature in hot-arid climate

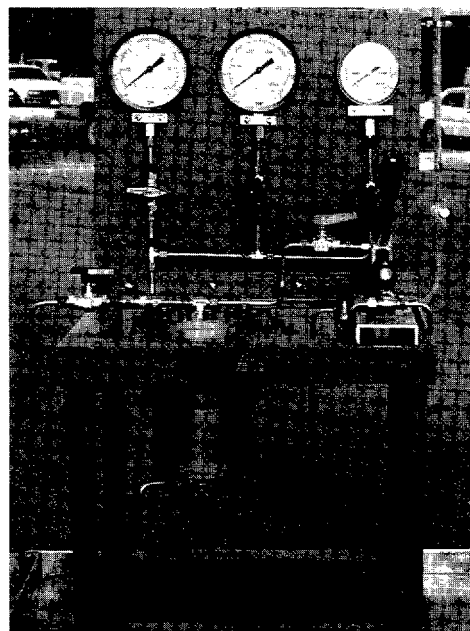


Fig. 5—High pressure permeability apparatus



Fig. 6—Typical specimens used to evaluate the corrosion cracking time of concrete after exposure to cyclic heat-treatment



Fig. 7—Typical specimens sawn from 4-in. cubes to evaluate the corrosion cracking time of concrete after exposure to cyclic heat-treatment

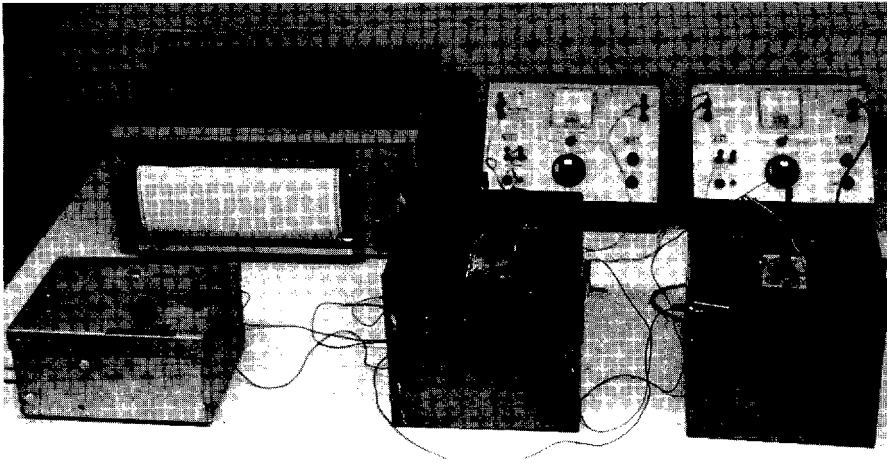


Fig. 8—Instrumentation for accelerated corrosion testing to evaluate corrosion cracking time

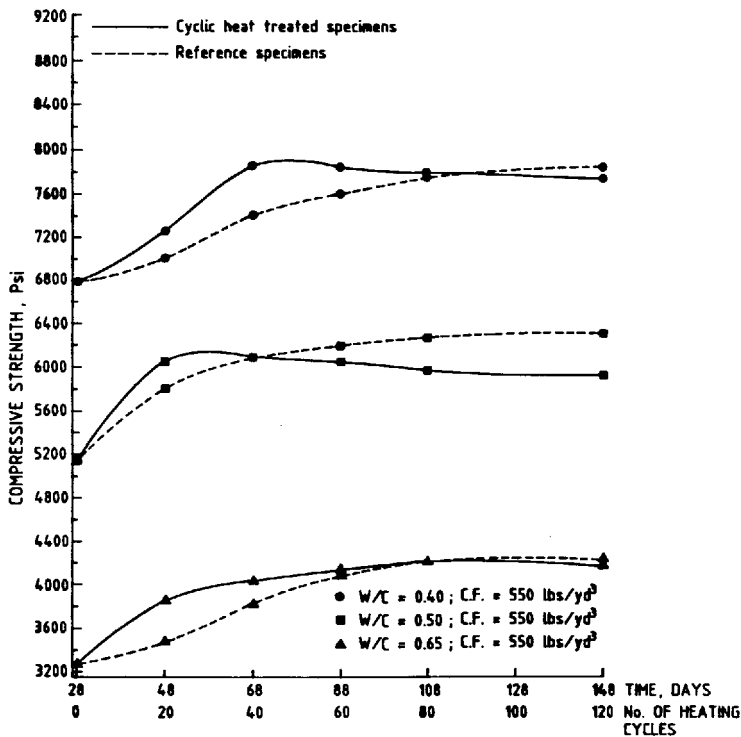


Fig. 9—Effect of cyclic heating on the compressive strength of concrete made with different water cement ratios

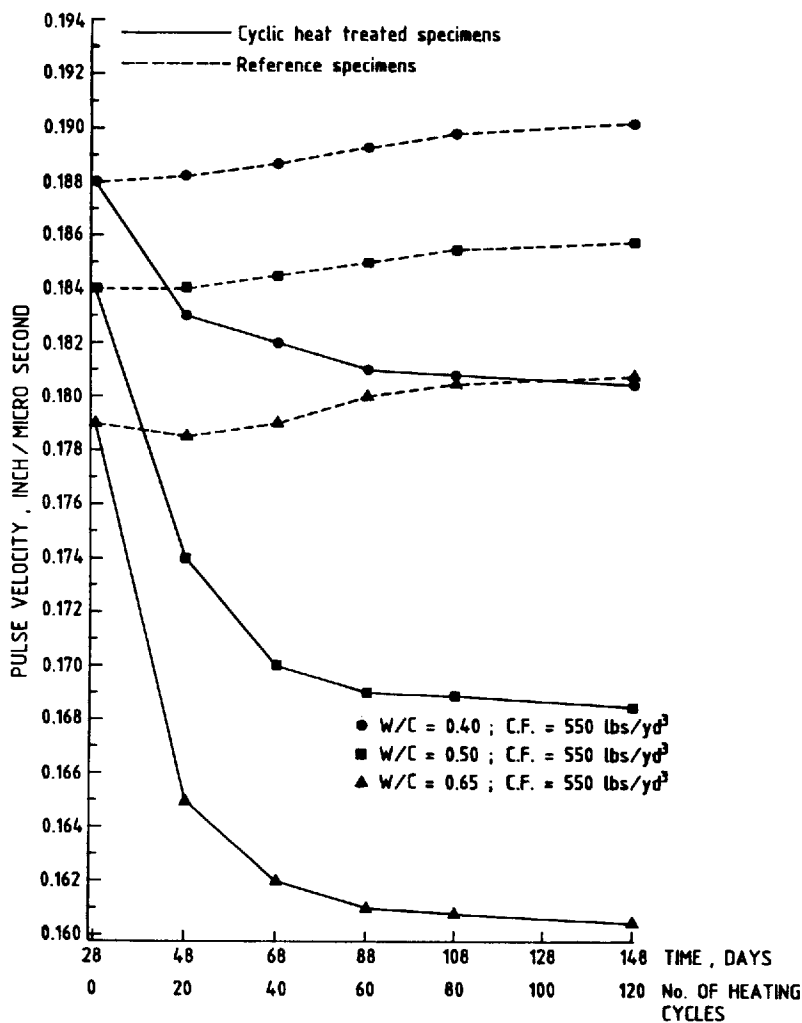


Fig. 10—Pulse velocities of reference and cyclic heat treated specimens made with different water cement ratios

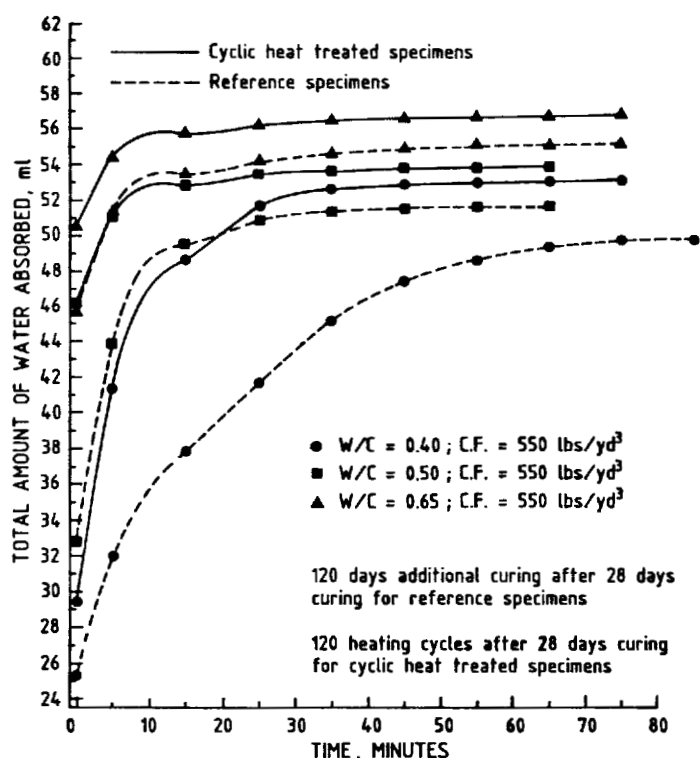


Fig. 11—Water absorption data for reference and cyclic heat treated specimens made with different water cement ratios

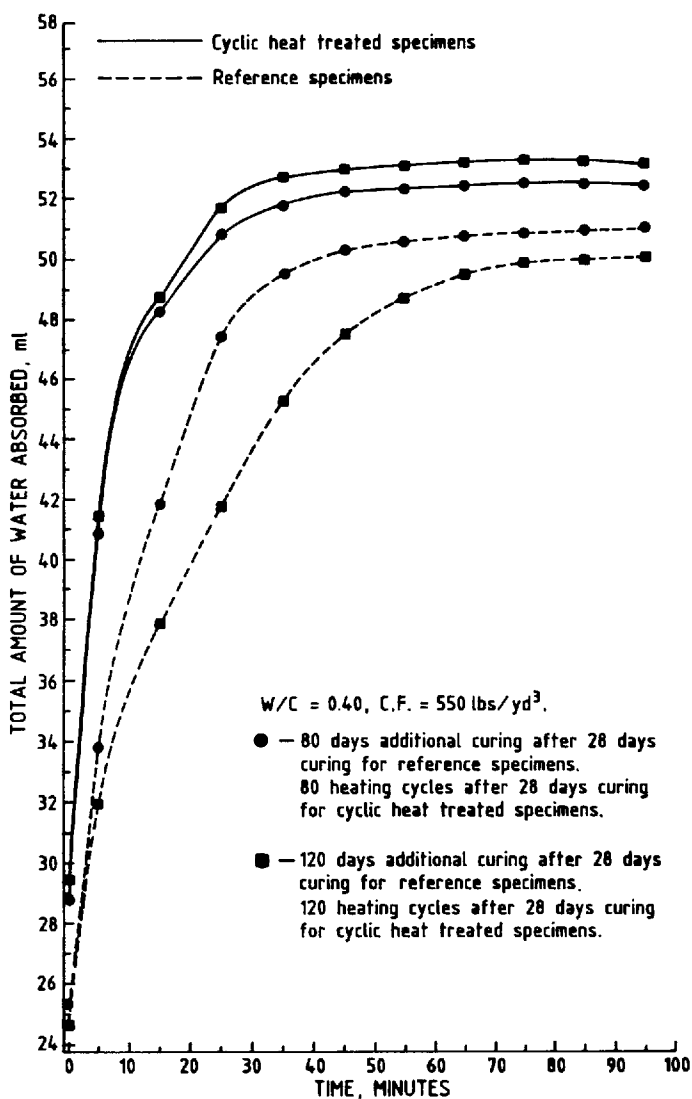


Fig. 12—Permeability of reference and cyclic heat treated specimens after 80 to 120 heating cycles

SP 139-2

Effect of Temperature on Cathodic Protection Criterion for Reinforced Concrete Structures

by Rasheeduzzafar and M.G. Ali

Synopsis: This paper presents the results of the effect of temperature on the cathodic protection (CP) level needed for effective control of chloride corrosion of reinforcing steel in concrete structures. The chloride levels in the concrete were 8 and 32 lb/yd³ (4.76 kg/m³ and 19 kg/m³) and chloride gradients were 1.5 and 2.0. Chloride gradient was created by embedding in the concrete specimen a relatively higher chloride-bearing macrocell and thereafter connecting the macrocell steel and the main steel through an external resistor. Current reversal technique was used to establish the protection level needed for effective control of reinforcing steel corrosion. Two sets of specimens were used: specimens of the first set (called reference specimens) were kept at the controlled room temperature of 25 °C (77 °F), and the specimens of the second set (called temperature-treated specimens) were kept in a temperature chamber having a peak value of 60 °C (140 °F). It was found that the corrosion activity of the reinforcing steel increases with an increase in the temperature to which concrete is exposed. Increased corrosion activity at a higher temperature exposure of 60 °C (140 °F), required an increased level of cathodic protection as indicated by higher protection current density, higher instant off protection potential and marginally higher decay potential at the beginning of the polarization period. The 60 °C (140 °F) temperature effect requires about 20 percent higher level of protection in terms of current density and about 20 to 30 mV higher instant off potential/decay potential for an initial polarization period of two months. Thereafter no additional protection is required against the temperature effect. The subsequent reduction in the level of cathodic protection required at higher temperature is indicative of a dominant influence of the electromigration factor in the interactive relationship between increased corrosion activity and the beneficial effect of electromigration of ions caused by the higher temperature.

Keywords: Cathodic protection; chlorides; concrete structures; corrosion; reinforced concrete; reinforcing steels; temperature

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INTRODUCTION

In the last decade cathodic protection has emerged not only as a strong corrosion control and prevention technique but also as a most viable rehabilitation method for rebar corrosion damaged concrete structures. It is the only appropriate method for the rehabilitation of reinforced concrete structures where deterioration has been caused by chloride induced corrosion of the reinforcement (1-4). All other rehabilitation techniques for chloride contaminated concrete are only cosmetic and do not control corrosion on a long term basis. Therefore, cathodic protection offers an attractive option to replacement, massive concrete removal or a continuous program of patch repairs throughout the life of a structure.

The current practice in terms of criteria for the cathodic protection of steel in concrete is entirely based on past experience with the protection of steel in soil or in seawater. The protection current requirements for reinforcing steel in concrete would be controlled by numerous factors. One of these factors is the ambient temperature to which concrete is exposed during service. High diurnal and seasonal variations of temperature constitute a significant feature of environment in the Gulf region where chloride corrosion of reinforcing steel is a serious concern (5-6). At higher temperatures the corrosion rate of steel in concrete as well as electromigration of ions at cathodic sites may be significantly

different than those in a cold weather regime. The prevalent CP criteria have been formulated mainly for cold weather conditions, typical of the United States of America, and may not be suitable for controlling corrosion of steel in concrete exposed to hot weather conditions in the Middle East, the sunbelt of U.S.A., Mexico or the Far East. To apply CP technique effectively for corrosion control of reinforcing steel in concrete exposed to hot weather conditions, the conventional CP criteria need a reevaluation incorporating the effect of ambient temperature. In typical summer months, an ambient temperature of 40 °C (104 °F) is a common occurrence in Gulf countries (6). A maximum temperature of 60 °C (140 °F) which included the effect of solar radiation on concrete surfaces (7), along with typical diurnal variation prevalent in the Middle east was simulated in a specially designed temperature chamber. An attempt has been made in this study to measure the shift in static potential, the protection potential, and the protection current requirement when the temperature is raised from the room exposure to 60 °C (140 °F) simulating typical temperature conditions on concrete surfaces in summer months.

SPECIMENS, INSTRUMENTATION AND TESTING

The test program for incorporating the temperature effect on CP requirement is shown in Table 1. The chloride contents of the specimens were 8 and 32 lb/yd³ (4.76 and 19 kg/m³) and the chloride gradients were 1.5 and 2.0. The chloride gradient in concrete was created by embedding a relatively higher chloride bearing macrocell in concrete, and thereafter connecting the macrocell steel and the main structural steel through an external resistor. The chloride gradient in a specimen is defined as the ratio of chloride levels in the macrocell concrete and concrete of the test specimen. For example, when the chloride level in the concrete of the specimen is 8 lb/yd³ (4.76 kg/m³) and the macrocell chloride content is 16 lb/yd³ (9.5 kg/m³), the chloride gradient is 2.0 (= 16.0/8.0; 9.5/4.76).

Test Specimens

Fig. 1 shows a typical specimen for evaluating cathodic protection criteria in hot weather conditions. All specimens were 18 x 12 x 3 in. (457 x 305 x 75 mm) slabs having the arrangement of reinforcement shown in Fig. 1. The main reinforcement comprises 1/2 in. (12 mm) diameter, 20 in (508 mm) long bars with 16 in (406 mm) length embedded in concrete. Three coupon bars were tied with the main steel bars. One cross bar, 3/8 in. (8 mm) diameter and 9 in (228 mm) long, interconnects the three main steel bars thereby providing electrical continuity between them. In each specimen, three macrocells described in the following section were used. Macrocells were cast prior to the casting of the test specimen and were positioned in the mold one day before casting of the test specimen.

The macrocell steel and the main steel were at the same level. At least three specimens were used for a particular chloride level and a particular temperature. The average value obtained from three replicates is used in the results. Concrete specimens were cast using a Type V portland cement with a w-c ratio of 0.56, cement content of 550 lb/yd³ (327 kg/m³) and a coarse to fine aggregate ratio of 1.43. Coarse aggregate was crushed limestone with a bulk specific gravity of 2.42 and absorption of 3.8 percent. Beach sand with a specific gravity of 2.70 and absorption of 0.25 percent was used as fine aggregate. Aggregates were washed to remove contamination and dust. The ICI H8 Tectrode anode used was properly positioned during casting. A 1/2 in. (12.7 mm) concrete overlay for the anode was adopted in all specimens. The specimens were demolded one day after casting and were then covered with wet burlap for curing. Test specimens were kept inside the temperature chamber 7 days after casting (called temperature treated specimens) whereas a parallel group of reference specimens was exposed to controlled room temperature of $25 \pm 2^\circ\text{C}$ (called reference specimens).

Artificial Corrosion Macrocells

The macrocells were cast as small concrete cylinders, 2 in (50 mm) diameter x 3 in (75 mm) long, with a short section of rebar (80 mm long) centrally placed in each of the concrete cylinders. The size of the macrocell steel was the same as that of the main steel. The concrete mix used in casting the macrocell was exactly the same as that used in casting the test specimen. The probe (macrocell) was connected to the reinforcement through a known resistor. By measuring the voltage drop across the resistor, the magnitude and direction of the current flow between the probe and ground (reinforcing steel) can be determined. When the positive voltmeter lead is connected to the ground, a positive voltage reading will indicate that the probe is corroding (8). A negative sign will indicate that the net current flow is from probe to ground (i.e. the probe is receiving current from the anode system which is flowing back to the rectifier through the lead connecting the probe to the main steel).

Current-Supplying Unit

The current-supplying unit comprises a rectifier which is a normal DC source with multiple outlets and the capability to adjust the current according to the requirement. The circuitry arrangement for the current-supplying unit used in the test program is shown in Fig. 2. The main components of this instrument are a variac, a rectifier, a capacitor and a number of variable resistors. The variac provides a variable A.C. supply of 0 to 220 volts; the rectifier converts the A.C. supply to D.C. output; the capacitor acts as a low-pass filter to the DC output voltage, and the variable resistor adjusts the current supply to a particular specimen. The

negative terminal of the D.C. supply is connected to the reinforcing steel, whereas the positive terminal is connected to the anode system. The ammeter, connected in series with the specimen and the variable resistor, measures the current impressed on the steel for each specimen. Current supply to the individual specimen is controlled by the variable resistor connected in series with this specimen. When the required amount of current could not be supplied to the steel in a specimen, even with the maximum adjustment of the potentiometer (variable resistor), the variac was used to change the supply voltage.

Temperature Chamber

A temperature chamber, capable of attaining a peak temperature of 60 °C (140 °F), is used in the test program to simulate the concrete surface temperature obtainable in the Arabian Gulf region. The temperature inside the chamber varied with time according to a predetermined regime which simulates the actual diurnal variation occurring in a typical summer day at Dhahran, Saudi Arabia. Fig. 3 shows the idealized temperature regimes which were expected to be attained and which were actually attained inside the temperature chamber. The temperature inside the chamber is communicated through a thermostat to a strip chart recorder which records the

temperature variation in the chamber. Wires from the specimens were taken out through a small hole provided in the chamber.

Potential Wheel Data Logger for Potential Measurement

A potential wheel and data logger used to monitor the steel potential values comprises a data bucket, capable of storing a sizable amount of potential data, and a printer. A hand held terminal connected to the data bucket is used to signal the data bucket for various operations. The potential wheel houses a silver-silver chloride (Ag/AgCl) reference cell. The steel reinforcement and the potential wheel are connected to the data bucket by two separate lead wires. The potential wheel is moved on a wet concrete surface to obtain potential readings at various locations. The potential wheel used had a very high input impedance of 10^{10} Ohms so that the observed potential values are unlikely to be affected by the surface dampness and changes in concrete resistivity.

Establishment of CP Criteria for Adequate Protection

All specimens were supplied with CP current 14 days after casting. Some of the specimens were kept as controls (never received CP current). Before CP application, electrons flow from the macrocell to the main steel and the corrosion current flows from the main steel to the macrocell

steel. In this situation, a positive voltage drop across the known resistor was observed when the positive voltmeter lead was connected to the ground (main steel reinforcement). The specimens were then activated with impressed DC current and the positive terminal of the supply was connected to the anode, whereas the negative terminal was connected to the main steel. The positive voltage drop across the known resistor gradually decreased with the increase of the impressed current, and subsequently at a particular level of DC current supply the voltage drop across the resistor was reduced to zero. At the zero voltage drop conditions, no local corrosion current flows between the macrocell steel and the specimen steel. In this state, the current passing to the main circuit is the current required for adequate protection. The current supply was then switched off and the steel-to-electrolyte potential was measured immediately using the potential wheel data bucket instrument. The potential measured in this manner was recorded as instant off potential. The steel to electrolyte potential was again measured 4 hours after switching the system off. Potential wheel data bucket was used for this measurement also. The difference between the instant off potential and the potential measured 4 hours after switching off the CP current was recorded as the 4 hour decay potential. The potentials of steel in the control specimens (never received CP current) were also measured using the potential wheel. The difference between the instant off potential measured and the potential obtained from the corresponding control specimen was recorded as shift potential. The current passing to the circuit at the onset of protection divided by the surface of reinforcing steel is reported as the protection current density.

RESULTS AND DISCUSSION

Corrosion Activity

Table 2 shows the comparative voltage drop in the resistor and the static potential of steel for concrete reference specimens exposed to a controlled room temperature of 25°C and the test specimens exposed to a variable temperature regime having a peak value of 60 °C (140 °F) (Fig. 3). The voltage drop in the resistor, prior to the application of CP, was found to be higher in the temperature-treated specimens than that in the reference specimens exposed to the room temperature (25 °C; 77 °F). For example, for 8 lb (4.76 kg) chloride-bearing concrete coupled with 16 lb (9 kg) chloride-bearing macrocell, the voltage drop values were 5.84 and 7.55 mV for reference specimens and temperature-treated specimens respectively. The percentage increase in the voltage drop for temperature-treated specimens, in comparison with the reference specimens, was observed to be 29 percent for 8 lb (4.76 kg) chloride-bearing concrete. A similar position was observed for the 32 lb (19 kg) chloride-bearing specimen having the 48 lb (28.5 kg) chloride-bearing macrocell; the percentage increase in voltage drop due to the temperature treatment of the 32 lb

(19 kg) chloride-bearing specimens was observed to be 24 percent. The resistor connects the steel embedded in the high chloride-bearing macrocell to the steel located in the relatively low chloride-bearing specimen. The voltage drop in the resistor is the result of corrosion current flow between the macrocell steel (anode) and the specimen steel (cathode); a higher voltage drop in the resistor is indicative of a higher corrosion current. The increased voltage drop obtained in the temperature-treated specimens, in comparison to the reference specimens exposed to room temperature (25 °C), is attributable to an increased corrosion activity at the steel surface when concrete environment is characterized by a higher temperature regime. The static potential of steel, prior to the application of CP, was found to be higher (more negative) in the temperature-treated specimens in comparison with the reference specimens. The average static potential value obtained in temperature-treated test specimens was 31 mV higher than the value obtained in reference specimens for 8 lb (4.76 kg) chloride-bearing specimens coupled with 16 lb (9.5 kg) chloride-bearing macrocell. For 32 lb (19 kg) chloride-bearing specimens coupled with 48 lb (28.5 kg) chloride-bearing macrocell the corresponding increase in static potential value (absolute value) was 25 mV. Fig. 4 shows the variation of static potential of steel with activation ages. The static potential of steel varied up and down with activation age; however, the variation was marginal. The higher static potential value in temperature-treated specimens, in comparison to the reference specimens, is ascribable to an increased corrosion activity at the steel surface (9,10) when concrete exposure environment is characterized by a higher temperature regime.

Both the voltage drop data and the static potential data indicate an increased corrosion activity at the steel surface when the concrete is subjected to a higher temperature regime.

Required Protection Level

Table 3 shows the comparative protection levels required for the reinforcing steel in concrete specimens which are exposed to a constant room temperature of 25°C and those which are exposed to a variable temperature regime having a peak value of 60°C. The instant off potential values needed to protect the reinforcing steel in the temperature-treated specimens were marginally higher compared to the values needed to protect the reinforcing steel in the reference specimens exposed to room temperature. For example, the instant off protection potential values observed in 8 lb (4.76 kg) chloride-bearing specimens having 16 lb (9.5 kg) chloride-bearing macrocells were 502 and 530 mv CSE for the reference (25 °C; 77 °F) and the temperature-treated (60 °C; 140 °F maximum) specimens respectively. For 32 lb (19 kg) chloride-bearing specimens having 48 lb (28.5 kg) chloride-bearing macrocell, the corresponding values were 608 mV and 628 mV respectively. The data obtained in both 8 lb (4.75 kg) and 32 lb (19 kg) chloride-bearing

specimens indicate only marginal increase in the instant off potential value due to higher temperature exposure. At least, the effect of significantly different corrosion activity in the temperature-treated specimens, as shown by the voltage drop data, is not reflected by a commensurate change in the instant off potential value.

Like the instant off potential values, the 4-hour decay potential values needed to protect the steel in temperature-treated specimens were also marginally higher, in comparison to the values needed to protect steel in the reference specimens. For example, the 4-hour decay potential values observed in 8 lb (4.76 kg) chloride-bearing specimens coupled with 16 lb (9.5 kg) chloride-bearing macrocells were 70 and 80 mV for the reference (25 °C; 77 °F) and the temperature-treated (60 °C; 140 °F maximum) specimens respectively. The corresponding values for 32 lb (19 kg) chloride-bearing specimens having 48 lb (28.5 kg) chloride-bearing macrocell were 60 and 66 mV respectively for the reference and the temperature-treated specimens. These results indicate that the effect of significantly higher corrosion activity in the temperature-treated specimens is not truly reflected in the 4-hour decay potentials. The influence of increased corrosion activity at higher temperature is not reflected in the shift potential value also. The shift potential values of 73 and 70 mV for the reference (25 °C; 77 °F) and temperature-treated specimens respectively were observed for the 8 lb (4.76 kg) chloride-bearing specimens. For the 32 lb (19 kg) chloride-bearing specimens, the corresponding values were 65 and 60 mV respectively.

The current density needed to protect the steel in temperature-treated specimens was found to be higher in comparison with the value required for the reference specimens. For example, the current densities for the 8 lb (4.76 kg) chloride-bearing specimens were 2.06 and 2.54 ma/sq ft (22.2 and 27.3 ma/sqm) for the reference (25 °C; 77 °F) and the temperature-treated (60 °C; 140 °F maximum) specimens respectively. The corresponding values for the 32 lb (19 kg) chloride-bearing specimens were 2.68 and 3.18 ma/sq ft (28.8 and 34.2 ma/sqm) respectively for the reference and temperature-treated specimens. As explained earlier, higher corrosion activity at higher ambient temperature caused a higher corrosion current to flow through the resistor which connects the macrocell steel and the specimen steel. As the function of the CP current is to reverse this local corrosion current flow, a higher CP current density would be required in the case of temperature-treated specimens to reverse the higher corrosion current generated.

Variation of the Protection Level Required with Time

Fig. 5 shows the instant off potential needed at different activation ages. Data presented in this Figure show, that at the beginning of activation, higher instant off potential values are needed for the temperature-

treated specimens in comparison to the reference specimens exposed to the room temperature (25 °C; 77 °F). Since the instant off potential value is dependent on the static base potential, the higher instant off potential obtained in the temperature-treated specimens, in comparison with the reference room temperature exposed specimens, is attributable to the increased static potential of steel observed at higher temperature. Data presented in Fig. 5 also show that at a later activation age, the instant off potentials needed for the temperature-treated specimens were lower (lower absolute value) in comparison with the reference specimens.

Fig. 6 shows the 4-hour decay potential needed at different ages for the temperature-treated specimens and the reference specimens. A marginal up and down variation of the 4-hour decay potential values with activation period was observed.

Fig. 7 shows the shift potential values needed at different activation ages for temperature-treated specimens as well as for reference specimens. There appears to be no effect of temperature on the shift potential needed for protection as indicated by the data of Fig. 7.

Fig. 8 shows the current density required at different activation ages for the temperature-treated specimens and reference specimens. As in the case of reference specimens, the current density needed for temperature-treated specimens also decreases with activation period. The temperature-treated specimens needed a higher current density for current reversal and cathodic protection. This higher current density would cause an increased electromigration of chloride ions from near the steel surface to regions away from steel in the temperature-treated specimens. The phenomena of electromigration of ions due to the application of CP current has been studied experimentally by the authors and is fully described in two recent publications (11,12). Cathodic protection tests carried out on chloride contaminated concrete showed that for 2 lb (1.19 kg) and 8 lb (4.76 kg) chloride-bearing concretes, reductions in chloride concentrations of 69 and 51 percents were obtained with a CP current density of 50 ma/ft² (538 ma/m²). The initial chloride concentrations of 0.40 and 1.30 percent by weight of cement were thus reduced to 0.124 and 0.61 percent by weight of cement for the 2 lb (1.19 kg) and 8 lb (4.76 kg) chloride bearing concretes respectively. The after-treatment chloride level (0.124 percent by weight of cement) for 2 lb (1.19 kg) chloride bearing concrete is reduced below the threshold chloride content (0.15 percent, ACI specification), whereas for the 8 lb (4.76 kg) chloride bearing concrete it is reduced to half its original value which may fall below the threshold value after a longer activation period. Also, a significant build up of sodium and potassium ions was observed at the steel-concrete interface as a result of CP application. Both the removal of chlorides and the degree of accumulation of (Na⁺ + K⁺) were a function of the impressed current density.

Enhanced electromigration of chlorides due to a higher current density would result in the natural or base potential shift to a more positive value requiring a lesser level of protection.

As shown in Fig. 8, the current density needed for the protection of the temperature-treated specimens became equal to that needed for the reference specimens after approximately two months of activation period. A decreased level of current density required at the later activation age in the temperature-treated specimens, in comparison to the reference room temperature exposed specimens, is attributable to a decrease in corrosion activity at the steel surface due to greater electromigration of chloride ions from the reinforcing steel vicinity caused by a higher current density in the temperature-treated specimens.

Interactive Effect of Higher Corrosion Activity and Electromigration of Ions at Higher Temperature with Time

The data presented above show two distinct effects of higher temperature on corrosion activity and cathodic protection. **First**, a higher corrosion activity occurs at the steel surface when the concrete is exposed to a higher temperature; **second**, an increased electromigration of ions takes place in higher temperature-treated specimens in comparison to the reference room temperature exposed specimens. Due to a higher impressed current in temperature-treated specimens, there will be increased electromigration of both chloride and alkali ions. This results in two concomitant advantages: (i) electromigration of chloride ions away from the steel results in lowering the corrosion activity at the steel surface, (ii) increased migration of alkali ions to steel surface results in an enhanced build-up of alkalinity at the steel-concrete interface (11,12). Both these happenings cause a shift of base potential to a more positive value. The interactive effect of higher corrosion activity and increased electromigration of ions in concrete exposed to a relatively higher temperature controls the protection level required. The increased corrosion activity warrants a higher protection level, whereas the increased electromigration of ions requires a lower protection level. A decreased level (more positive) of instant off potential needed at the later activation age for temperature-treated specimens, in comparison with the reference room temperature exposed specimens, indicates that the effect of the enhanced electromigration of ions overtakes the effect of the increased corrosion activity. A decreased value of current density at a later activation period, for both the temperature-treated and reference specimens, is attributable to the reduction of corrosion activity near the steel caused by the electromigration of chloride ions from the vicinity of the steel surface. As explained for the instant off potential value, a decreased level of current density required at the later activation age for the higher temperature-treated specimens, in comparison with the

reference room temperature exposed specimens, indicates that the effect of the increased electromigration of ions exceeded the effect of the increased corrosion activity. However, the effect of the above stated interactive action of higher corrosion activity and increased electromigration at the higher temperature could not be confirmed for the 4-hour decay potential and the shift potential data presented in this study.

CONCLUSIONS

1. A higher static potential as well as a higher intensity of corrosion current observed for concrete exposed to a higher temperature of 60 °C (140 °F) as compared to 25 °C (77 °F) indicate an increase in the corrosion activity with an increase in the ambient temperature to which concrete is exposed.
2. Increased corrosion activity at the higher temperature of 60 °C (140 °F) required an increased level of cathodic protection as indicated by higher protection current density, higher instant off protection potential and marginally higher decay potential at the beginning of the polarization period. The temperature effect requires about a 20 percent higher level of protection in terms of current density and about 20 to 30 mV higher instant off potential/decay potential for an initial polarization period of two months. Thereafter no additional protection is required for the temperature effect.
3. As polarization continues, a higher protection current density required for higher temperature (60 °C; 140 °F) exposure condition results in an increased electromigration of ions. This electromigration of ions provides a two-fold benefit in reducing corrosion activity. **Firstly**, chloride ions move away from the steel-concrete interface reducing chloride attack on steel, and **Secondly**, alkali cations move toward the location of reinforcing steel resulting in an enhanced build-up of alkalinity at the steel-concrete interface. Both these effects cause a shift of base potential to a more positive value. The reduction in corrosion activity is confirmed by a reduction in the required level of protection with the progress of polarization. The level of protection at 60 °C (140 °F) after 2 months was observed to be lesser than required for exposure at 25 °C (77 °F) as well as the protection required at the beginning of the activation period.

The reduced level of cathodic protection required for concrete exposed to a higher temperature is indicative of a dominant influence of the electromigration factor in the interactive relationship between increased corrosion activity and the beneficial effect of electromigration of ions caused by the higher temperature.

4. The data presented in this paper indicate that significant benefits in reducing long term cathodic protection requirement could be derived by adopting a higher level of protection in terms of higher protection current/instant off/decay potentials in the initial period of activation. The longer is the period of this enhanced protection, the greater would be the reduction in the corrosion activity due to the beneficial effect of electromigration of ions, eventually requiring reduced protection on a long term basis. It is also possible that in low and moderate conditions of chloride corrosion, with an initial high level of protection for a finite period, the corrosion activity may stop completely.

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TABLE 1 — TEST PROGRAM FOR DETERMINING THE EFFECT OF TEMPERATURE ON CATHODIC PROTECTION CRITERION

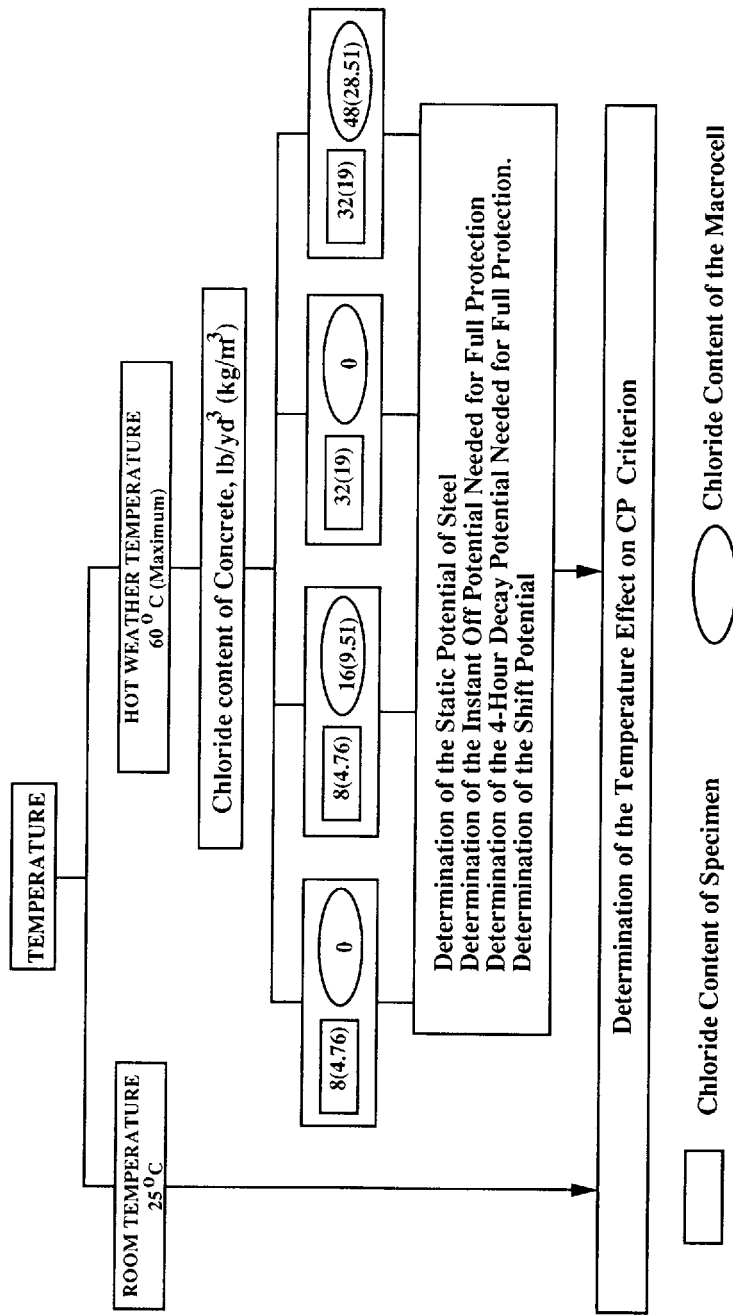
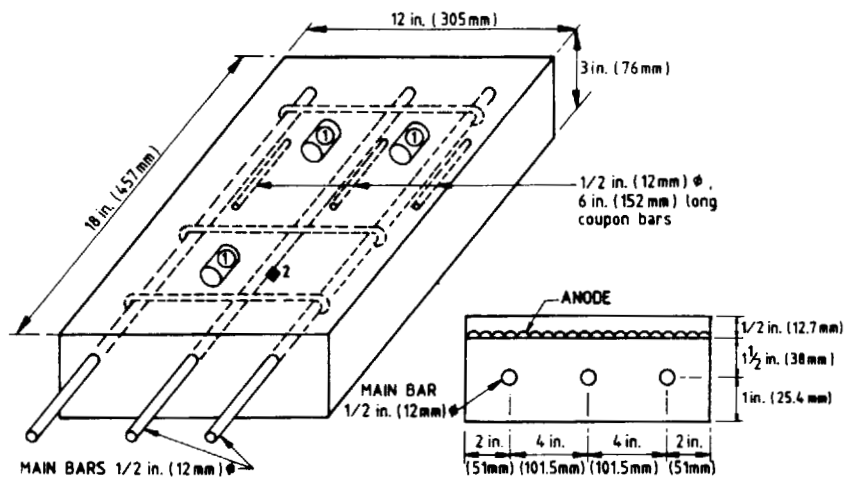


TABLE 2 — COMPARISON OF THE VOLTAGE DROP IN THE MACROCELL AND THE STATIC POTENTIAL OF STEEL AT ROOM TEMPERATURE EXPOSURE (25 C) AND HIGHER TEMPERATURE EXPOSURE (60 C) CONDITIONS

Chloride Content of the Specimen, lb/yd ³ (kg/m ³) of Concrete	Macro-cell Cl Content kg/m ³ of Concrete	Voltage Drop Across the Resistor (mV)		Static Potential of Steel, mV, CSE	
		At 25°C	At 60°C	At 25°C	At 60°C
0.8 (4.76)	16 (9.5)	+5.84	+ 7.55	429	460
32 (19)	48 (28.5)	+4.80	+ 5.95	543	568

TABLE 3 — COMPARISON OF THE PROTECTION LEVEL REQUIRED FOR REINFORCING STEEL IN CONCRETE EXPOSED TO ROOM TEMPERATURE (25 C) AND HIGHER TEMPERATURE (60 C)

Chloride Content of the Specimen, lb/yd ³ (kg/m ³) of Concrete	Macro-cell Cl Content lb/yd ³ (kg/m ³) of Concrete	Instant off Potential mv, CSE		Four-Hour Decay Pot-ential, mv		Shift Potential mv		Current Density at Steel Surface ma/sq ft	
		At 25°C	At 60°C	At 25°C	At 60°C	At 25°C	At 60°C	At 25°C	At 60°C
08 (4.76)	16 (9.5)	502	530	70	80	73	70	2.06	2.54
32 (19)	48 (28.5)	608	628	60	66	65	60	2.68	3.15



- ① Location of Macrocell
- ② Location of Corrosion Probe

Fig. 1—Typical specimen used in the study of high temperature effect on cathodic protection criterion

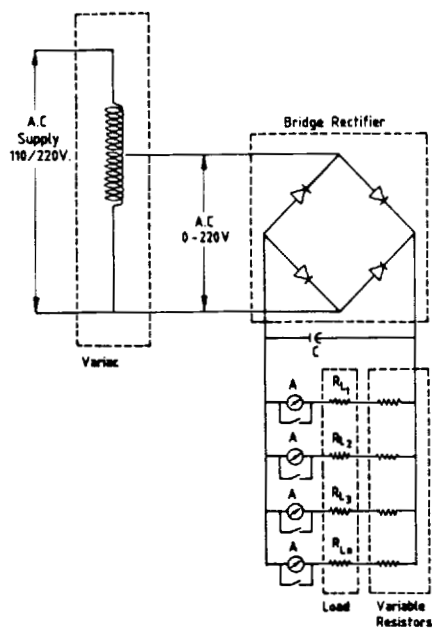


Fig. 2—Electrical schematic for impressing current on reinforcing steel in concrete specimens

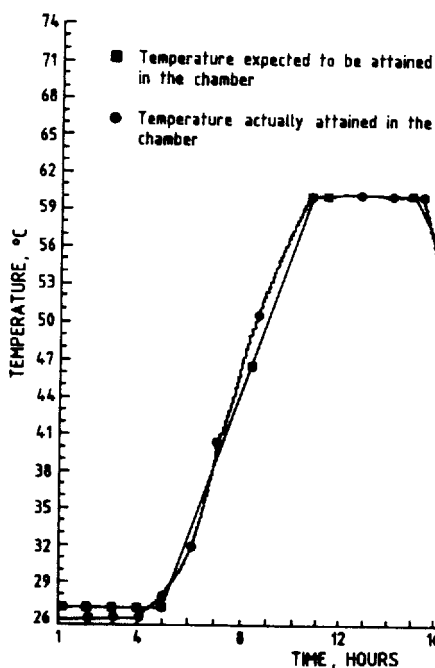


Fig. 3—Temperature expected and actually attained in the temperature chamber

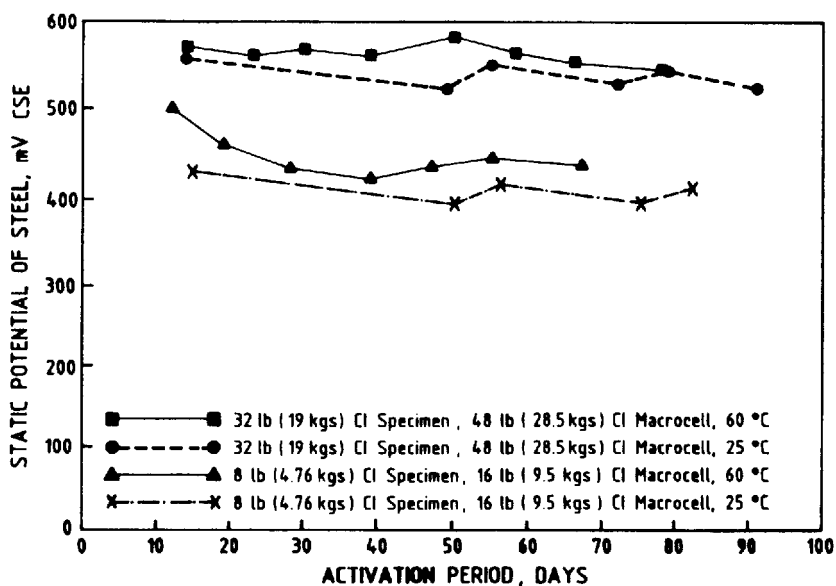


Fig. 4—Effect of ambient temperature on static potential of steel

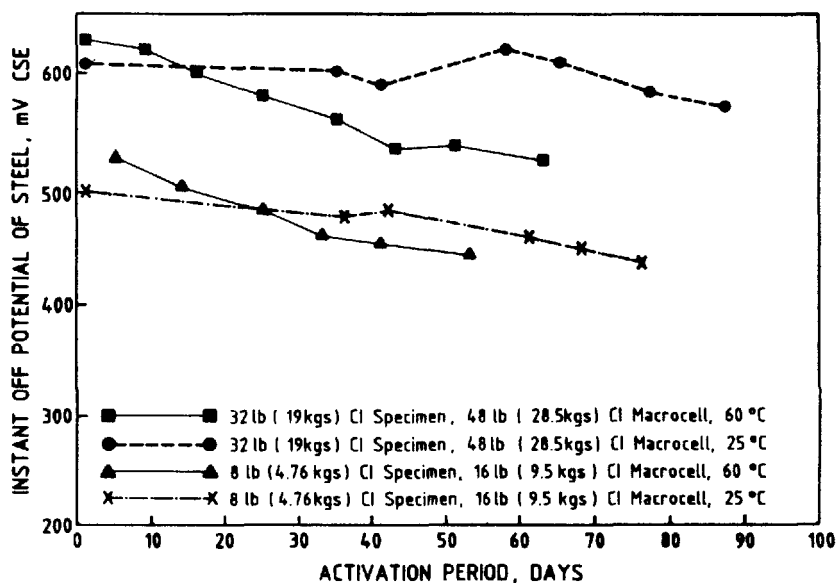


Fig. 5—Effect of ambient temperature on instant off potential needed for protection

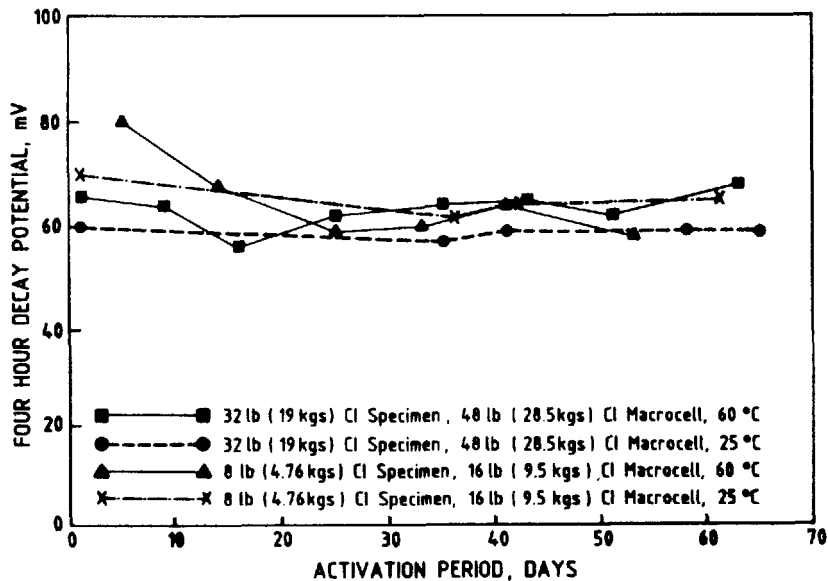


Fig. 6—Effect of ambient temperature on four hour decay potential needed for cathodic protection

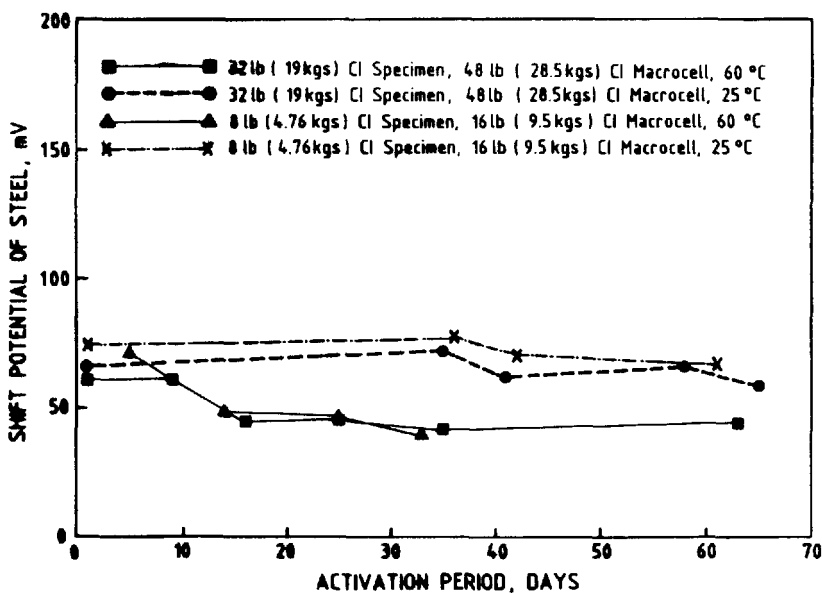


Fig. 7—Effect of ambient temperature on shift potential needed for cathodic protection

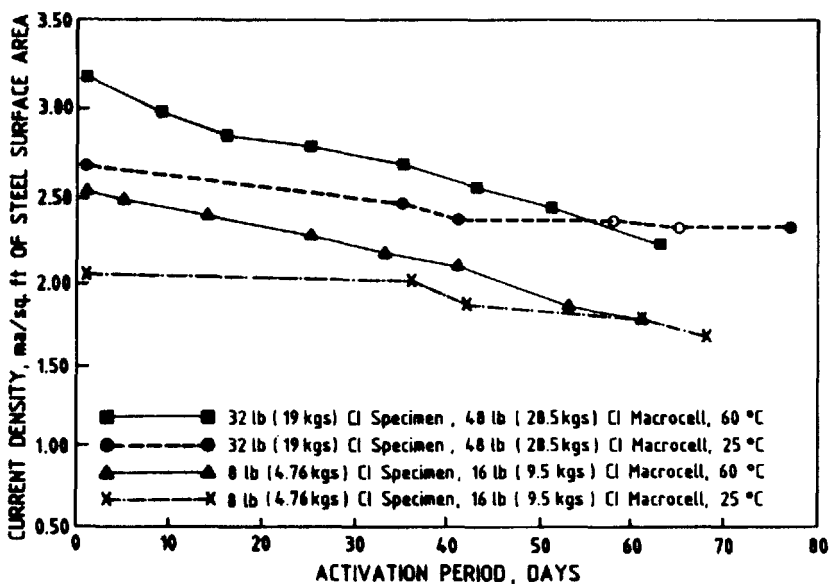


Fig. 8—Effect of ambient temperature on current density needed for cathodic protection

SP 139-3

Electron-Optical Evaluation of Concrete Cured at Elevated Temperatures

by B.A. Clark, E.A. Draper, R.J. Lee,
J. Skalny, M. Ben-Bassat, and A. Bentur

Synopsis: Laboratory concrete made under different curing conditions was evaluated using electron-optical techniques. Differences in microstructure and strength were observed in relationship to the water-to-cement ratio, wet/dry curing, temperature of curing, presence of supplementary materials, and mode of preheating. This presentation highlights the partial results of the microstructural evaluation.

Keywords: Cracking (fracturing); curing; durability; microstructure; porosity; strength; temperature; water cement ratio

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INTRODUCTION

"Concrete mixed, transported, and placed under conditions of high temperature, low humidity, or wind requires an understanding of the effects such environmental factors have on concrete properties and construction operations" [1]. Hot weather effects include increased water demand, shorter setting times, increased rate of slump loss, increased tendency for plastic cracking, decreased strength and durability, etc. The effects are, in addition, influenced by such variables as the presence of admixtures, quality of the cement, and the mixing equipment used.

Cracking of concrete, as well as other consequences of

high-temperature curing, are to a large degree the result of changes in the microstructure of the cement matrix [2,3], and the resulting damage to concrete structures is more often than not caused by a combination of high temperature curing along with other mechanisms of deterioration [4].

An issue of considerable present interest is the effect of curing temperature on the subsequent formation of secondary ettringite, also referred to as delayed ettringite formation (DEF), and its possible influence on cracking of steam cured concrete elements [e.g., 5-7].

The present study concentrated on the microstructure of several concrete mixes cured at four different temperatures. Heat treatment affects the microstructure and porosity of concrete, and these effects may influence the final properties of concrete. There is limited published evidence showing that high-temperature curing leaves some "fingerprints" on the microstructure [8,9].

The goal of the present investigation was to better define the observable microstructural features, as revealed by scanning electron microscopy (SEM) with energy dispersive x-ray spectroscopy (EDS), that may result from the differences in maximum curing temperature and other variables, particularly wet and dry postcuring. If better understood and controlled, microstructural engineering could result in improved concrete strength and durability. Better understanding of microstructural features could also lead to improved appreciation of the importance of proper curing and help in determining the causes of concrete deterioration.

EXPERIMENTAL

Four different concrete mixes were prepared. As shown in Table I, the mix variables were the cement, sand and aggregate ratio, presence of fly ash or condensed silica fume with high-range water-reducer (HRWR, superplasticizer), and water-to-cement ratio. Samples were prepared to have the same slump.

The aggregate used was a dolomitic limestone (65% CaCO_3), having an apparent density of 2750 kg per cubic meter. The dolomitic limestone had an observed maximum dimension of 8 mm and was typically sub-angular and equant to elongate. Both fine- and coarse-grained examples of this phase were present.

The sand consisted predominantly of quartz, with minor amounts of potassium feldspar (microcline) and plagioclase feldspar. The sand had a density of 2620